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ARCH DAMS: STRESS STUDIES FOR ROSS AND DIABLO DAMS

Joe T. Richardson,* A.M. ASCE, and Owen J. Olsen**
(Proc. Paper 1267)

SYNOPSIS

This paper presents representative results of a long-time study of stress from strain meters at Ross Dam. Included is an outline of the method followed in deriving stress from embedded strain meters. Also presented are the preliminary results obtained from a stress study on the Diablo Dam by strain relief measurements.

FOREWORD

This paper is one of a group presented at the ASCE Symposium on Arch Dams, June, 1956, at Knoxville, Tennessee.

Since the last symposium on masonry dams (April, 1939), much progress has been made in the design and construction of arch dams and their appurtenances. This Symposium was planned to enable engineers concerned with arch dams to exchange their ideas and experiences for the benefit of all.

At this time, it is not known exactly how many papers will be printed from the Symposium. So far, seventeen papers have been approved: "Arch Dams: Their Philosophy," by Andre Coyne (Proc. Paper 959); "Arch Dams: Trial Load Studies for Hungry Horse Dam," by R. E. Glover and Merlin D. Copen (Proc. Paper 960); "Arch Dams: Portuguese Experience with Overflow Arch Dams," by A. C. Xerez (Proc. Paper 990); "Arch Dams: Theory, Methods, and Details of Joint Grouting," by A. Warren Simonds (Proc. Paper 991); "Arch Dams: Santa Giustina Single-Curvature Arch Dam," by Claudio Marcello (Proc. Paper 992); "Arch Dams: Measurements and Studies on Santa Giustina Dam," by Claudio Marcello (Proc. Paper 993); "Arch Dams: The Reno Di Lei Double-Curvature Arch Dam," by Claudio Marcello (Proc.

Note: Discussion open until November 1, 1957. Paper 1267 is part of the copyrighted Journal of the Power Division of the American Society of Civil Engineers, Vol. 83, No. PO 3, June, 1957.

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Paper 994); "Arch Dams: Isolato Double-Curvature Arch Dam," by Claudio Marcello (Proc. Paper 995); "Arch Dams: Rio Freddo Dam with Gravity Abutments and Cut-Offs," by Claudio Marcello (Proc. Paper 996); "Arch Dams: Design and Observation of Arch Dams in Portugal," by M. Rocha, J. Laginha Serafim, and A. F. da Silveira (Proc. Paper 997); "Arch Dams: Development in Italy," by Carolo Semenza (Proc. Paper 1017); "Arch Dams: Design of the Kamishiiba Arch Dam," by C. C. Bonin and H. W. Stuber (Proc. Paper 1018); "Arch Dams: Observed Behavior of Several Italian Arch Dams," by Dino Tonini (Proc. Paper 1134); "Arch Dams: Measurements and Studies of Behavior of Kamishiiba Dam," by H. Kimishima and C. C. Bonin (Proc. Paper 1182); "Arch Dams: Construction of the Kamishiiba Arch Dam," by K. M. Mathisen and C. C. Bonin (Proc. Paper 1183); "Arch Dams: Review of Experience," by Robert E. Glover (Proc. Paper 1217); and "Arch Dams: Stress Studies for Ross and Diablo Dams," by Joe T. Richardson and Owen J. Olsen (Proc. Paper 1267).

INTRODUCTION

As a part of this symposium on arch dams there are presented representative results of the long-time study of stress from strain measurements made at Ross Dam. The measurements were obtained by Carlson elastic-wire strain meters embedded in the mass concrete of the structure. The study of the stress behavior of this massive structure has been a joint undertaking between the City of Seattle, Department of Lighting, and the Bureau of Reclamation. The former organization contributed instrumental equipment and their installation in the dam, materials for required laboratory creep determination tests, and the program of systematically obtaining data. The latter organization contributed technical assistance during instrument installation, conducted laboratory tests for the determination of creep and other concrete properties, and performed the complex computations required to derive stress from values of measured data combined with the laboratory-derived function of creep. The results of the stress study were to be used[1] as would materially benefit both organizations.

Ross Dam

Ross Dam shown in Figure 1, is a variable radius arched structured located on the Skagit River in Northern Washington and is ranked as one of the world's highest arch dams. It has a height of 540 feet, a crest length of 1,300 feet, and contains 905,000 cubic yards of concrete. The dam is constructed in 26 sections, separated by contraction joints. Each section is 50 feet wide at the axis of the dam. During construction, the sections were raised systematically in a manner that provided for alternate low, intermediate, and high sections.

The dam was constructed in two major stages. The first stage was entirely completed in 1940, with the top of the dam at elevation 1365 except for sections at each abutment that had been left lower to serve as spillways. The later stage was completed between 1943 and 1949, with the top of the dam

1. Simonds, A. Warren, "The Determination of Stresses in an Arch Dam from Observed Strains," RILEM Paper 4 U.S.A., Themes 1b et 3a October 1955.

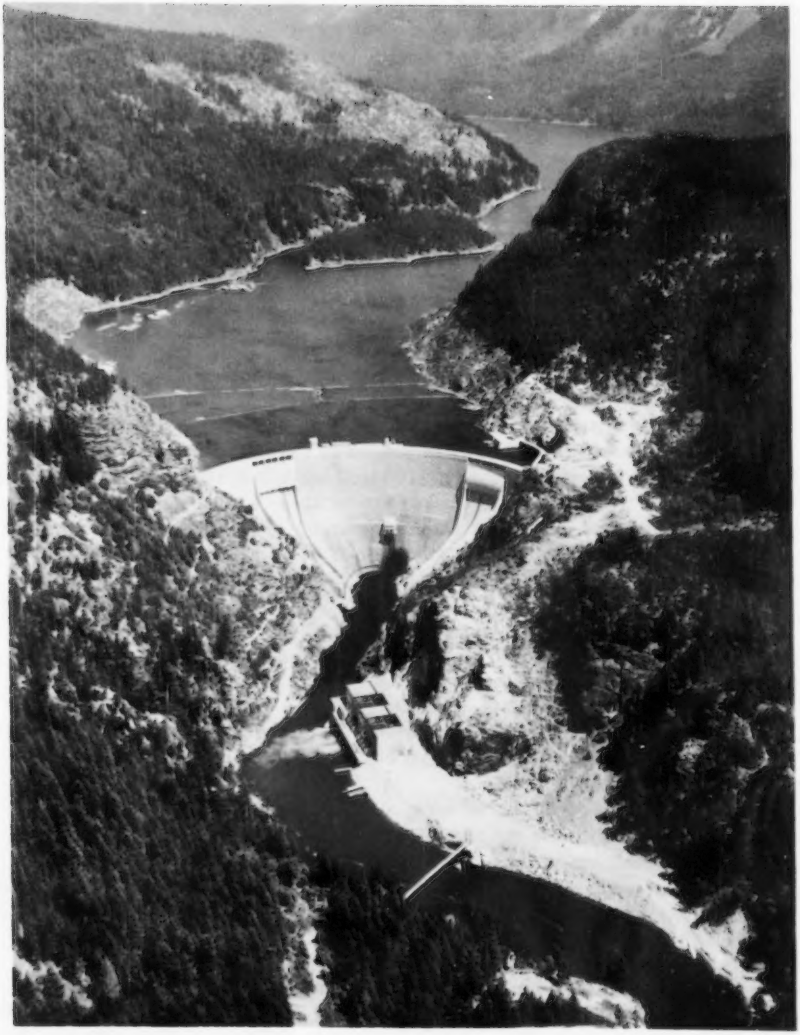


Figure 1. Ross Dam and Reservoir.

being at its present height, elevation 1615. No artificial cooling was done in the first stage concrete; artificial cooling was done in the later stage concrete.

The sectional construction of the dam provided for keyed contraction joints which were radial to the upstream face and approximately radial to the downstream face, the tangents being connected by the arc of a circle.

Ross Dam is unusual in some of its structural features. These include the provision for additional construction beyond the present stage to raise the dam to its ultimate height, elevation 1733.0. This contemplated additional construction will require thickening the entire dam, beginning at the base. To form a keyed bond between the old and the new concrete sections, patterns of square and vertical keys were cast in the concrete at the downstream face.

Other unusual features are the two hooded spillways incorporated in the ends of the dam. Their design was determined from experimental scale models built in the Bureau of Reclamation Laboratories in Denver. The flows of water routed down the two spillways collide in midstream, partially dissipating their energy against each other. The spillway deflector hoods are built of concrete, heavily reinforced with steel.

Construction progress and reservoir water surfaces elevation for the structure are shown on Figure 2.

Strain Meter Layout

Groups of strain meters were installed at 98 selected locations in the mass concrete of the dam as shown in Figure 3.

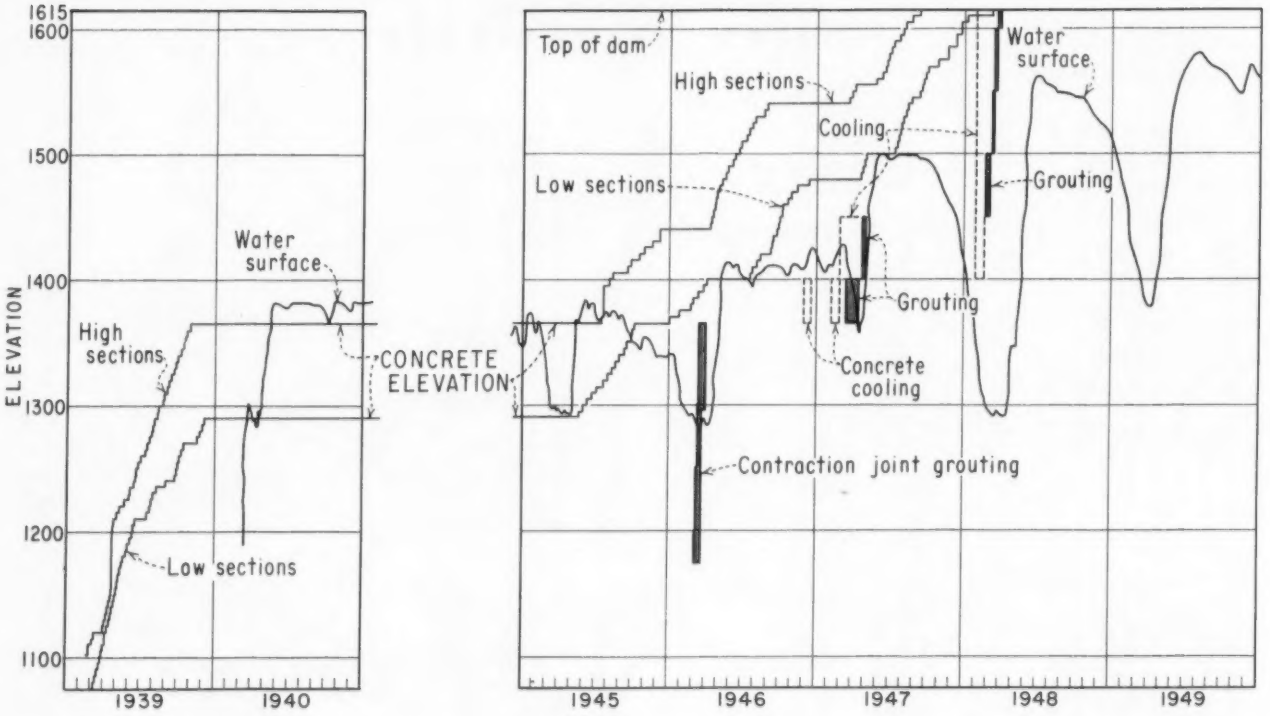
The groups of strain meters were placed in the planes of 11 arches, spaced 50 feet vertically between elevation 1100 and elevation 1600; additional groups of strain meters were placed in the planes or arches at elevations 1225 and 1275. In the planes of arches between elevations 1150 and 1350, the strain meter groups were generally located on lines parallel to the joint near the crown and parallel to the foundation at the abutment; in the planes of arches between elevations 1400 and 1600, strain meter groups were included on two to four intermediate lines, in addition to the lines near the crown and abutment. Each line of strain meter groups included a group near each face of the dam; most lines included one or more groups at the interior of the section; few lines contained only face groups.

Each group of strain meters is composed of several instruments oriented in the directions indicated in the detail on the lower part of Figure 3. Duplicate meters were not included.

With many groups of strain meters, a single additional strain meter was included to furnish data for determining growth or shrinkage in the mass concrete. The instrument was cast into a concrete block contained in a cardboard-lined wooden box. The whole assembly was embedded in the mass concrete of the dam with a strain meter group. This single strain meter thus served as a stress-free instrument contained in the mass concrete of the dam.

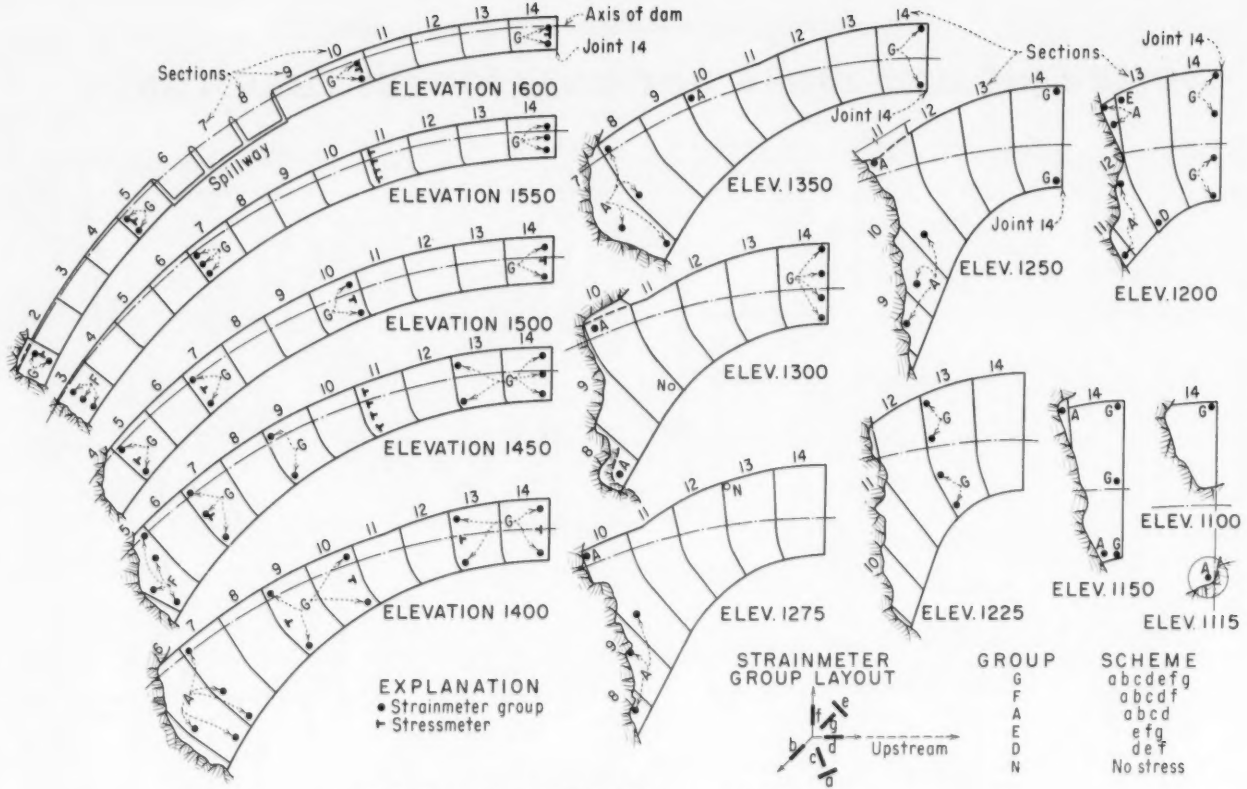
On several sections of the upper arches, stress meters were embedded to determine stress in the direction of arch thrust. A total of 21 stress meters was embedded in the structure. Results from 5 of these instruments are included with the stress results.

Since the dam is approximately symmetrical about the line of centers, strain meters were installed only in the right half.



CONSTRUCTION PROGRESS AND RESERVOIR OPERATION

Figure 2. Construction Progress and Reservoir Operation.



STRAINMETER LAYOUT

Figure 3. Strain Meter Layout.

Laboratory Tests

The derivation of stress from the collection of strain meter observations made at the dam required a knowledge of the properties of the concrete used in the structure. Accordingly, during the later stages that measurements were made at the dam, tests were conducted on concrete specimens in the Bureau of Reclamation laboratories in Denver.

Creep specimens were made in the laboratory[2] with Ross Dam materials shipped from the project. These specimens of identical concrete comprised 3 series, totaling 21 cylinders, each cylinder containing a Carlson strain meter.

In addition to the creep specimens, 30 cylinders were cast and tested at various ages to determine compression strength, modulus of elasticity, and Poisson's ratio.

All creep cylinders were sealed in lightweight copper jackets to prevent loss of moisture, thus approximating the curing condition of mass concrete. Sustained load of the cylinders was maintained in some cases by heavy coil springs and in others by a hydraulic diaphragm or ram.

A logarithimic function was found from the laboratory data which defined creep in Ross Dam concrete at each age of loading. The equation is as follows:

$$\epsilon = \frac{1}{E} + F(K) \ln (t+1)$$

in which:

ϵ is elastic + creep strain per psi
E is modulus of elasticity, varying with age
F(K) is creep function, a constant for each particular age of loading
ln (t+1) is the natural logarithm of the time after loading + 1 day

The creep parameters for Ross Dam concrete are shown in the following table. By interpolation, values for the parameters could be found for any age of loading.

Loading Age in Days	$\frac{1}{E}$	(F)K
2	.480	.0898
7	.285	.0666
28	.245	.0326
90	.213	.0239
365	.180	.0170
5 year	.150	.0081

2. "Investigations of Creep Characteristics of Ross Dam Concrete," Bureau of Reclamation Concrete Laboratory Report No. C-787, Denver, Colorado, March 16, 1955.

The coefficient of thermal expansion was found from laboratory tests to be 4.6 millionths of an inch per inch per 1^{0} F. Poisson's ratio was found to be 0.20.

Method of Determining Stress

Essentially, stress has been derived from strain determined from the strain meters embedded in the structure combined with creep properties derived from measurements made on loaded cylinders of identical concrete in the laboratory. The strain in the structure in turn is derived from progressive differences of measured length change from each strain meter and the calibration constants of the strain meters.

Mathematically, strain is represented by an equation for a 3-dimensional system. Stress follows through application of a modification of Hooke's law.

Mathematical computations were carried out in the initial stages of the analysis by desk calculators, and in the later stages of analysis, by mechanical punched card computers (IBM).

The program of strain measurements at Ross Dam extended over a long period of time, during which time the concrete properties and the temperatures varied considerably. In the computations, successive corrections[3] were required for temperature in the concrete, for dilatation as measured by two combinations of three orthogonal strain meters, and for the Poisson's ratio effect.

The stress-strain relation can be expressed in the following equation which reflects the stress on orthogonal axes.

$$\sigma_x = E \cdot \left[\frac{1}{(1+\mu)(1-2\mu)} \left((1-\mu)\varepsilon_x + \mu(\varepsilon_y + \varepsilon_z) \right) \right]$$

and in like manner for σ_y and σ_z, in which

σ is stress
ϵ is strain
E is instantaneous elastic modulus
μ is Poisson's ratio
x, y, and z are three orthogonal axes

Due to the Poisson's ratio effect in elastic materials, the strain measured on any axis is due not only to the stress acting on that axis, but also to the stress on the orthogonal axes. For the stress computation, it has been found convenient to adjust the strain on a given axis so that only the stress acting along that axis is represented. The adjusted strain is designated ϵ' and is computed from the three orthogonal strains from the equation:

$$\varepsilon'_x = \frac{1}{(1+\mu)(1-2\mu)} \left[(1-\mu)\varepsilon_x + \mu(\varepsilon_y + \varepsilon_z) \right]$$

3. Raphael, Jerome M., "The Development of Stresses in Shasta Dam," Transactions ASCE, Vol. 118, 1953, p. 289.

and in a lime manner for ϵ'_y and ϵ'_z. Stress is then computed from strain and elastic modulus from the equation:

$$\sigma_{xyz} = E\,\varepsilon_{xyz}$$

Since the properties of concrete do not remain constant when subjected to sustained loading and lapse of time, necessity requires modification of the Hooke's law equation to allow for creep in the concrete. Thus:

$$\sigma_{xyz} = E'\varepsilon'_{xyz}$$

where E' represents the average elastic modulus of concrete during the period ϵ' is developing and changes with the age of the concrete.

Some strain meter groups contained instruments only in the plane of the arch. At these locations, synthesis of strain in the vertical direction was required to determine 3-dimensional stress. The synthesized vertical strain was approximated, using the vertical strain from other nearby strain meter groups. The strain meter groups that furnished the comparable vertical strain had horizontal strains of similar trend and magnitude to the horizontal strains at groups that required the synthesized values.

Initially all computations were performed manually using electric desk calculators. This method[4] was a tedious and time-consuming process. Several successive improvements were made in the computational process. A method was devised by which stress computation was considerably accelerated by using IBM punched cards and high-speed digital computers.[5] The machine process at the beginning, merely reproduced mechanically the operations that had formerly been performed by hand. This computation may be represented by the equation:

$$\Delta\sigma = \frac{\Delta\varepsilon}{\frac{1}{E'} + F(K)\,\ln\,(t+1)}$$

Where:

$\Delta\sigma$ is the change in stress during an increment of time

$\Delta\epsilon$ is the total measured strain at the end of the increment of time, minus the elastic strain plus the creep strain at the beginning of the increment of time.

4. Jones, K., "Calculation of Stress from Strain in Concrete," Bureau of Reclamation, Technical Memorandum No. 653, February 1955.
5. Raphael, Jerome M. and Bruggeman, John R., "Analysis of Strain Measurements in Dams by Use of Punched Card Machines," a paper presented at summer Convention ASCE, Denver, Colorado, June 1952.

$\frac{1}{E} + F(K) \ln (t+1)$ is the function of elastic and creep strain per psi obtained from the laboratory tests.

In the digital computer process, cards were punched for each increment of time for creep function and for total observed strain. The machine computers then integrated for each strain meter and for each increment of time, the elastic strain plus creep strain from all previous stress increments, subtracted that total from the observed strain, and computed the next stress increment. The cards were sorted and the process repeated for the next stress increment, and so on to the end of the study. A later modification of the computational method using an average logarithm concept was devised. This modification made possible the carrying forward in successive computations the summation of total strain and made unnecessary the computations of individual increments of creep for each increment of time.

In the earlier machine computation of stress, prior strain was found as the summation:

$$\varepsilon = \sum_{i=1}^{n-1} \Delta\varepsilon_i \text{, or } \varepsilon = \sum_{i=1}^{n-1} \left[\frac{1}{E'_{i+1}} + F(K)_{i+1} \ln(t+1) \right] \Delta\sigma_i$$

in which the interval i varies from 1 to n, n being the number of intervals. In later machine computation of stress, using the average logarithm concept, prior strain is expressed as:

$$\varepsilon = \sum_{i=1}^{n-1} \frac{1}{E_{i+1}} \Delta\sigma_i - \ln \operatorname{avg}(t+1) \sum_{i=1}^{n-1} F(K)_{i+1} \Delta\sigma_i$$

Stress Results

The stress results for the complete strain meter study cover the period beginning with meter placement, during construction, and extends through 1949, the cutoff year selected for the study. The study included all meters that were functioning in a satisfactory manner. Meter readings are still continuing as it is contemplated that a further possible extension of the analysis may be desirable at a later date. Figure 4 has been included to show as a sample the complete stress history for one group of strain meters.

Stress results have been selected for presentation at this symposium with the view of illustrating for several conditions of loading, the manner in which

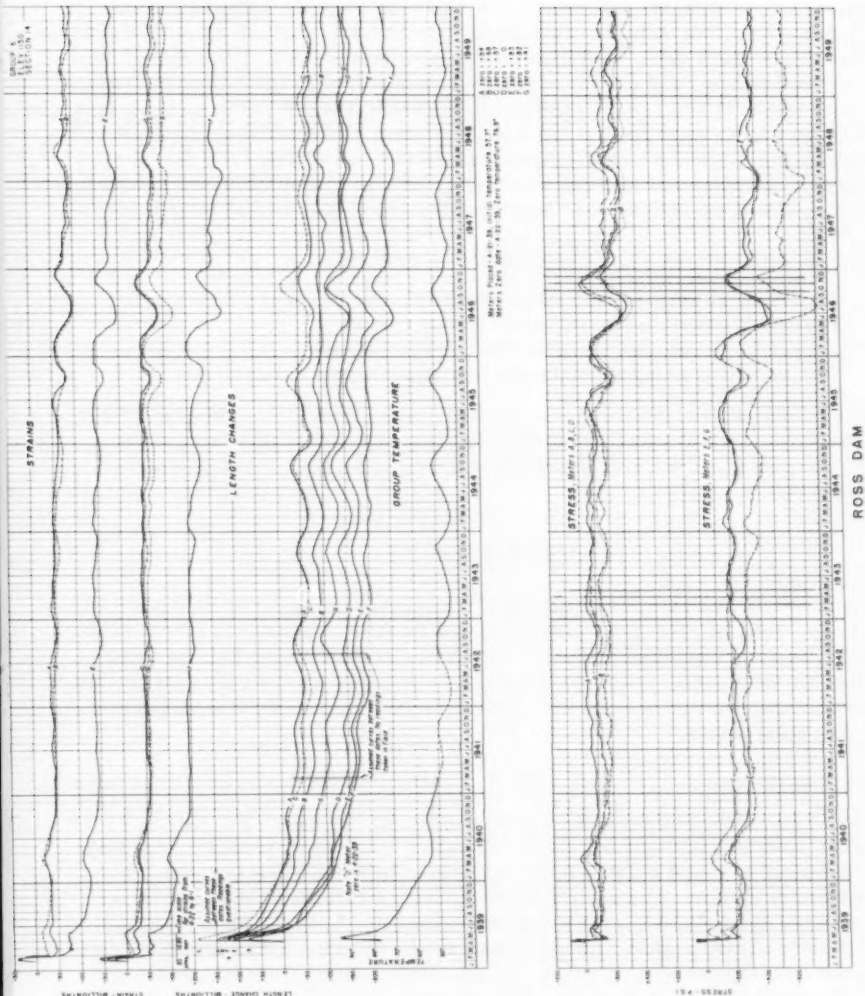


Figure 4. Sample Stress History from Group of Strain Meters.

vertical stress is developed in the cantilever, and horizontal stress is developed in the plane of the arch in the direction of arch thrust, along with available accompanying conditions of principal stress.

The selected stress results are for two arches located at elevation 1200 and elevation 1450. Sections at these elevations are on the crown cantilever, the arch abutments, and intermediate locations. The loading conditions for these representative stress results are those for January 1 of 1940, 1942, 1948, and 1950. The selected results correspond to conditions of partial load on the initial-stage dam, total load on the initial-stage dam, partial load on the completed dam, and partial normal operating load on the completed dam.

Since the first and second conditions of loading are for the dam completed to elevation 1365, only results from the elevation 1200 groups of strain meters can be included for the initial-stage dam. In the illustrations for the third and fourth conditions of loading, results for both elevation 1200 and elevation 1450 are shown, as the dam had been constructed to its present height, elevation 1615.

The selected stress results are shown on four figures.

Figure 5, shows vertical stress results
Figure 6, shows horizontal arch stress results
Figure 7, shows vertical principal stress results
Figure 8, shows horizontal arch principal stress results

These figures show progressively, for four loading conditions, the variation and increase in stress as developed at the two elevational locations in the crown cantilever, at several locations in one arch, and at two locations in another arch.

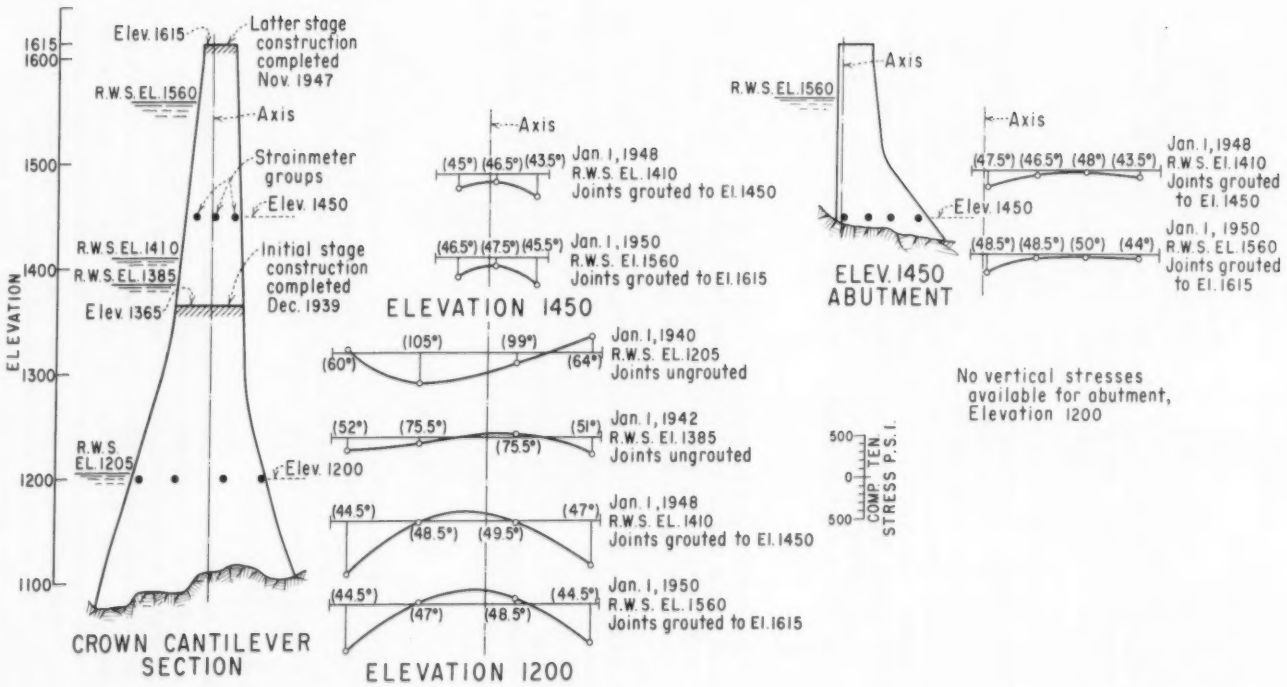
Vertical Stress

Figure 5 shows the vertical stress distribution at elevations 1200 and 1450. The major dead and live loads are indicated on the section of the crown cantilever. The height to which concrete had been placed for each selected date and the corresponding reservoir water surface elevation are indicated on this section. Temperature is indicated at each point of stress measurement for each condition shown.

Crown Cantilever

The stress distribution on the crown cantilever for January 1, 1940, at elevation 1200 indicates compressive stress in the central portion of the section, reducing to slight tension at the upstream face and greater tension at the downstream face. Temperatures at the interior of the section were greater than at the faces. Two years later, January 1, 1942, the interior compressive stresses had reduced in magnitude and the tensile stresses near the faces had become compressive. At all points temperature had reduced. For both conditions of loading the joints between blocks were ungrouted. For the later condition the reservoir water surface had increased to elevation 1385, being retained by a timber crib constructed on the top of the initial-stage dam. No cooling of concrete had been done in the initial-stage dam.

On January 1, 1948, after construction of the dam to its present height, elevation 1615, low compressive stress was indicated at the interior of the section, increasing in magnitude at each face. Temperatures at stress points


Elev. 1615
Latter stage construction completed Nov. 1947
R.W.S. EL. 1560
Axis
Strainmeter groups
Elev. 1450
R.W.S. EL. 1410
R.W.S. EL. 1385
Initial stage construction completed Dec. 1939
Elev. 1365
R.W.S. EL. 1205
Elev. 1200
ELEVATION
1615
1600
1500
1400
1300
1200
1100
CROWN CANTILEVER SECTION
ELEVATION 1450
(45°) (46.5°) (43.5°) Jan. 1, 1948 R.W.S. EL. 1410 Joints grouted to El. 1450
(46.5°) (47.5°) (45.5°) Jan. 1, 1950 R.W.S. El. 1560 Joints grouted to El. 1615
(105°) (99°) (60°) (64°) Jan. 1, 1940 R.W.S. EL. 1205 Joints ungrouted
(52°) (75.5°) (51°) (75.5°) Jan. 1, 1942 R.W.S. El. 1385 Joints ungrouted
(44.5°) (48.5°) (49.5°) (47°) Jan. 1, 1948 R.W.S. EL. 1410 Joints grouted to El. 1450
(44.5°) (47°) (48.5°) (44.5°) Jan. 1, 1950 R.W.S. EL. 1560 Joints grouted to El. 1615
ELEVATION 1200
500 0 500
COMP. TEN
STRESS P.S.I.
R.W.S. EL. 1560
Axis
Elev. 1450
ELEV. 1450 ABUTMENT
(47.5°) (46.5°) (48°) (43.5°) Jan. 1, 1948 R.W.S. El. 1410 Joints grouted to El. 1450
(48.5°) (48.5°) (50°) (44°) Jan. 1, 1950 R.W.S. EL. 1560 Joints grouted to El. 1615
No vertical stresses available for abutment, Elevation 1200

VERTICAL CANTILEVER STRESSES

Figure 5. Vertical Cantilever Stresses.

had further reduced from the temperatures shown for January 1, 1942. The joints had been grouted to elevation 1450. The reservoir water surface had increased to elevation 1410.

With elapse of 2 years, the stresses for January 1, 1950, show slight change at the interior. Tension appears at the interior and a slight reduction of compression is indicated near the faces. Temperature at stress points had reduced slightly. Joints had been grouted to the complete height of the dam. Reservoir water surface was at elevation 1560.

At elevation 1450 the vertical stress conditions are shown for the two latter conditions of loading that have been shown for elevation 1200. Cooling of concrete had been done on the later stage of the dam.

The stress distribution for January 1, 1948, indicates lesser compressive stress at the center of the section than near the faces. Temperature was uniform on the section and joints had been grouted to elevation 1450. The reservoir water surface was at elevation 1450, 40 feet below the points of stress measurement. Two years later, January 1, 1950, stresses remained approximately the same. Near the faces stress had increased slightly in compression. Temperature was approximately the same, and the contraction joints had been grouted to the full height of the dam.

Abutment

No vertical stresses are available for the abutment at elevation 1200. At the right abutment of the dam, elevation 1450, stress distribution on January 1, 1948, was compressive, being greatest near the upstream face. Reservoir water surface was below this elevation at that time. Temperature was approximately uniform through the section and grouting had been done to elevation 1450. On January 1, 1950, the stress distribution was approximately the same as previously shown. A slight increase was indicated near the upstream face and slight decreases indicated on the remainder of the section. The temperature within the section was approximately the same as for January 1, 1948. The reservoir water surface had increased to elevation 1560 and the joints had been grouted to the top of the dam.

Horizontal Stress

The horizontal stress distribution for arches at elevations 1200 and 1450 are shown in Figure 6. Arch stresses are greatly influenced by temperature variations, and this effect is strongly shown in the progressive stress history of the several sections through the two arches shown.

Elevation 1200 Arch

The arch stress distribution for the elevation 1200 arch is shown for the crown and the abutment. At the crown on January 1, 1940, compressive stresses were indicated at the interior of the section, while at the faces tension was indicated. Interior temperatures were considerably greater than temperatures near the faces. The reservoir water surface was at elevation 1205 and joints were upgrouted. Two years later the reservoir water surface had increased to elevation 1385, the joints still remaining upgrouted. The stress distribution at the arch crown indicates reduced compression at the interior of the section. One interior stress point had passed from

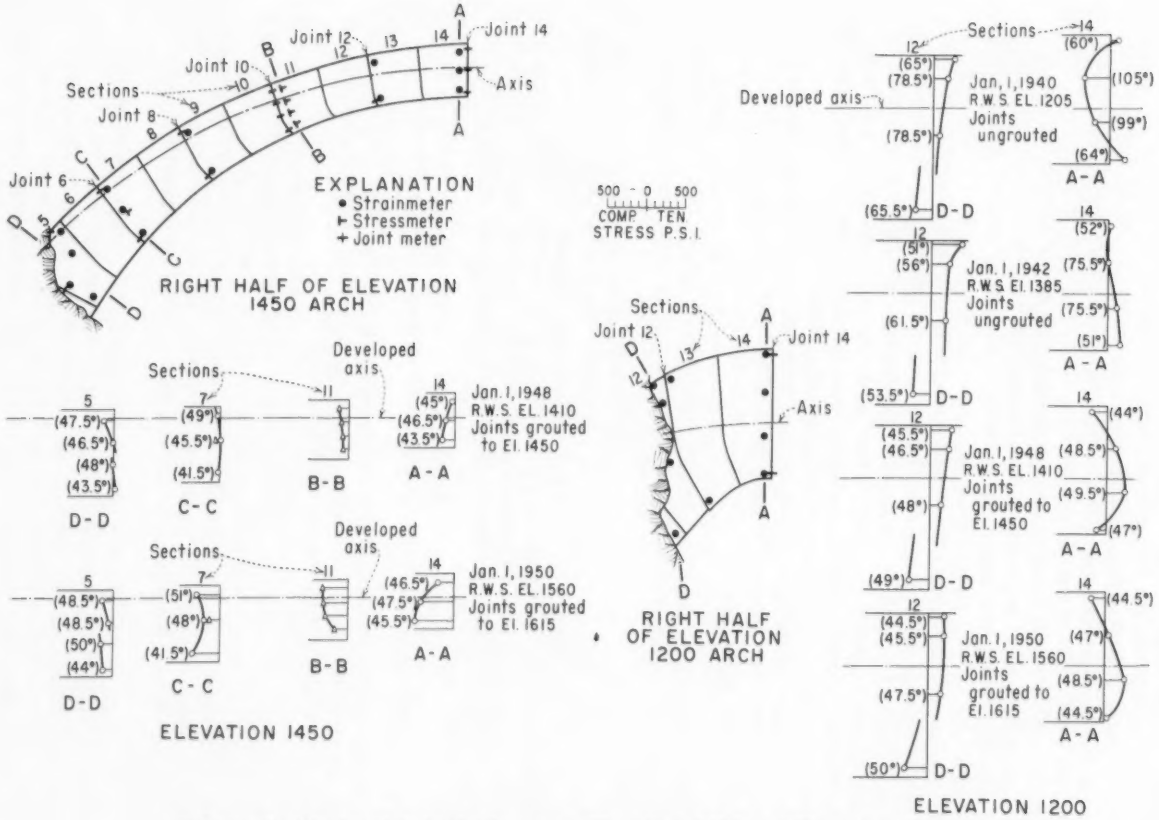


Figure 6. Horizontal Arch Stresses, Elevation 1450 and Elevation 1200.

compression to tension. The upstream portion of the section showed very little stress present, and near the downstream face the stress had reduced in tension. Temperatures at the interior of the section had decreased considerably while at the face the temperature had reduced only slightly.

On January 1, 1948, after completion of the dam to elevation 1615, the reservoir water surface was at elevation 1410. Joints had been grouted to elevation 1450. The stress distribution on the crown section of the elevation 1200 arch showed increased tension at the interior of the section and increased compression near the faces. Temperature on the section indicates an approximate uniform condition, reduced considerably below that shown in 1940 and 1942.

On January 1, 1950, with the reservoir water surface at elevation 1560 and the joints grouted to the top of the dam, the stress distribution indicated tension was still present at the interior of the section. Compressive stress remained at the upstream face and stress at the downstream face had reduced in compression and had become slightly tensile. Temperature conditions had remained very near the same as those shown 2 years earlier.

At the elevation 1200 arch abutment, the stress distribution indicates little change between the dates selected. Early stress distribution indicated tension over the entire upstream two-thirds of the section. The tensile stress increased with time at the point nearest the center of the section, then again decreased slightly. Near the upstream face tensile stress increased between 1940 and 1942, reduced in 1948, and in 1950 a further slight decrease was indicated.

Decreasing temperatures are indicated at all abutment stress points, becoming approximately stable in 1948 and 1950. The stress point nearest the downstream face at the abutment has not been included on the same stress curve as the points at the interior and upstream face of the section, since this stress point is not in the same block with the other three stress points. At this stress point compression has been indicated for the entire record shown here.

Elevation 1450 Arch

The stress distribution at several sections on the elevation 1450 arch are shown for the two latter conditions of loading after the present dam had been completed to its present height, elevation 1615.

On January 1, 1948, stress distribution at the crown indicated all stresses were compressive. Stress is greater at the center and downstream face than at the upstream face. At Section B-B, near the right quarter point of the dam, the stresses are from four stress meters. On January 1, 1948, greatest stress is near the upstream face. At the interior of the section and near the downstream face stress is less. On January 1, 1950, all stresses had increased in magnitude.

At Section C-C of Figure 6, near the abutment, the stress distribution indicates all stresses are compressive though low in magnitude. Near the center of Section C-C the stress from a single stress meter has been included for comparison with stress from the strain meter group. The stress from the stress meter is in good agreement with that from the strain meter group.

At the abutment a stress distribution is shown that is similar in shape to that at Section C-C. The highest compressive stress is indicated near the

upstream face. Here again, a break is shown in the stress distribution curve as the four strain meter groups are not on a straight line. Temperature at all sections of measurement indicate the same trend with values being approximately the same.

Two years later on January 1, 1950, the reservoir water surface had increased to elevation 1560 and the joints had been grouted to the top of the dam.

Stress distribution at the crown of the arch showed considerable increase in compression over that of 2 years earlier. The greatest stress was still indicated at the downstream face.

At Section B-B of Figure 6, near the quarter point of the dam, the stress meters indicated stress to be compressive, increasing between 1948 and 1950. The great stress magnitude was near the upstream face of the dam. At Section C-C considerable increase in compressive stress is shown with slightly greater stress near the downstream face than near the upstream face, each face stress being considerably greater than the stress at the center.

At the abutment increased compressive stress is indicated. Temperature conditions for the entire arch show little change over those of 2 years earlier, having remained approximately stable throughout the arch.

Principal Stresses

The orientation and magnitude of the principal stresses in the plane of the crown cantilever, elevations 1200 and 1450, for the selected conditions of loading are shown in Figure 7. Principal stress determination in the vertical planes at the abutments of the elevations 1200 and 1450 arches were not feasible since the abutment strain meter groups contained insufficient meters in the vertical plane to make the required computations.

The orientation and magnitude of the principal stresses in the planes of the elevations 1200 and 1450 arches for the selected conditions of loading are shown in Figure 8. In the plane of the elevation 1450 arch the principal stresses are shown for sections at the crown, the abutment, and several intermediate locations.

CONCLUSIONS

No attempt has been made to completely justify the development of the stresses shown from the study since there is lack of available measured supporting deflection and deformation data. These data are usually required to trace the trend of stress development that would be indicated by correlated external measurements. The stress illustrations herein presented represent through progressive application the effects of known quantities of load comprising concrete, reservoir water, temperature, and grouting effects.

Trial load stress studies made prior to the construction of the completed dam, when compared with the strain meter stress study, show expected differences in stress results. This is not surprising since the study of stress by strain measurements and the study of stress by analytical methods is each on a basis different from the other. Each method reflects the assumptions and conditions for which the particular analysis is made, and thus cannot be expected to show equal final results.

Oftentimes, the analytical study represents only the stress conditions for

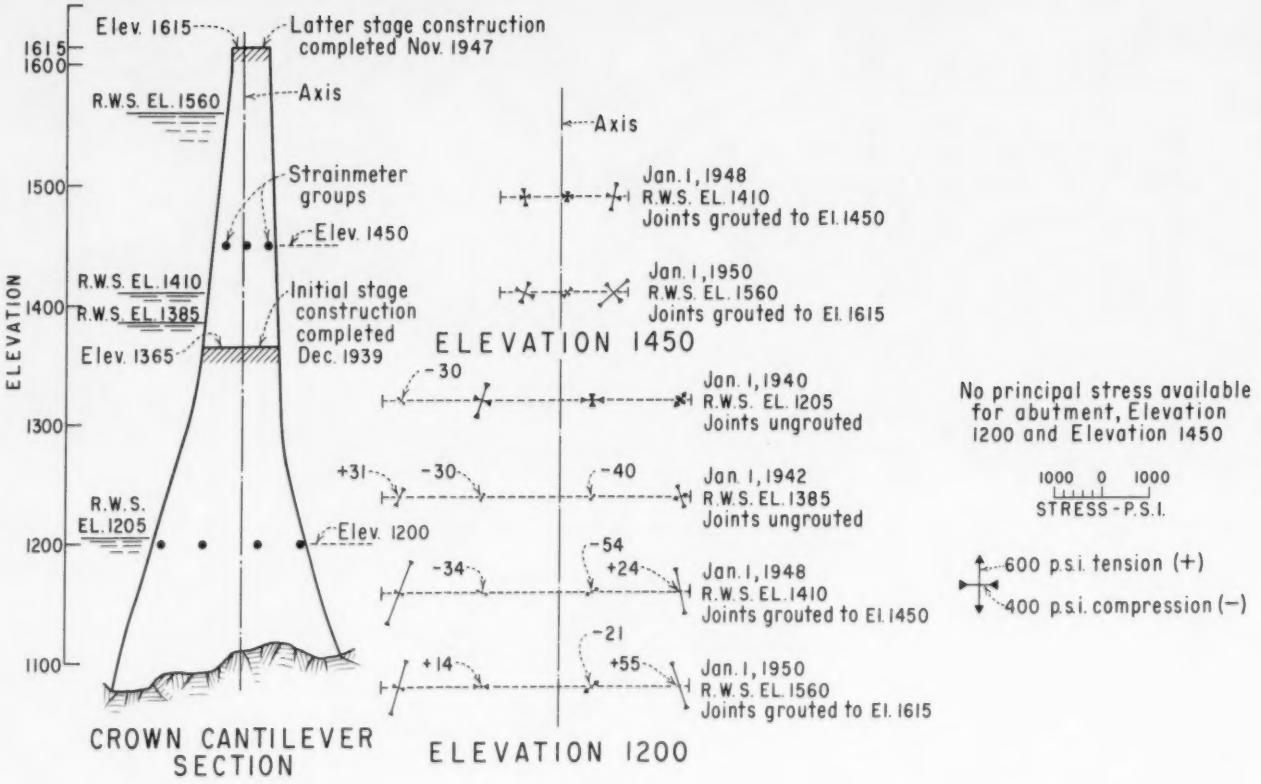
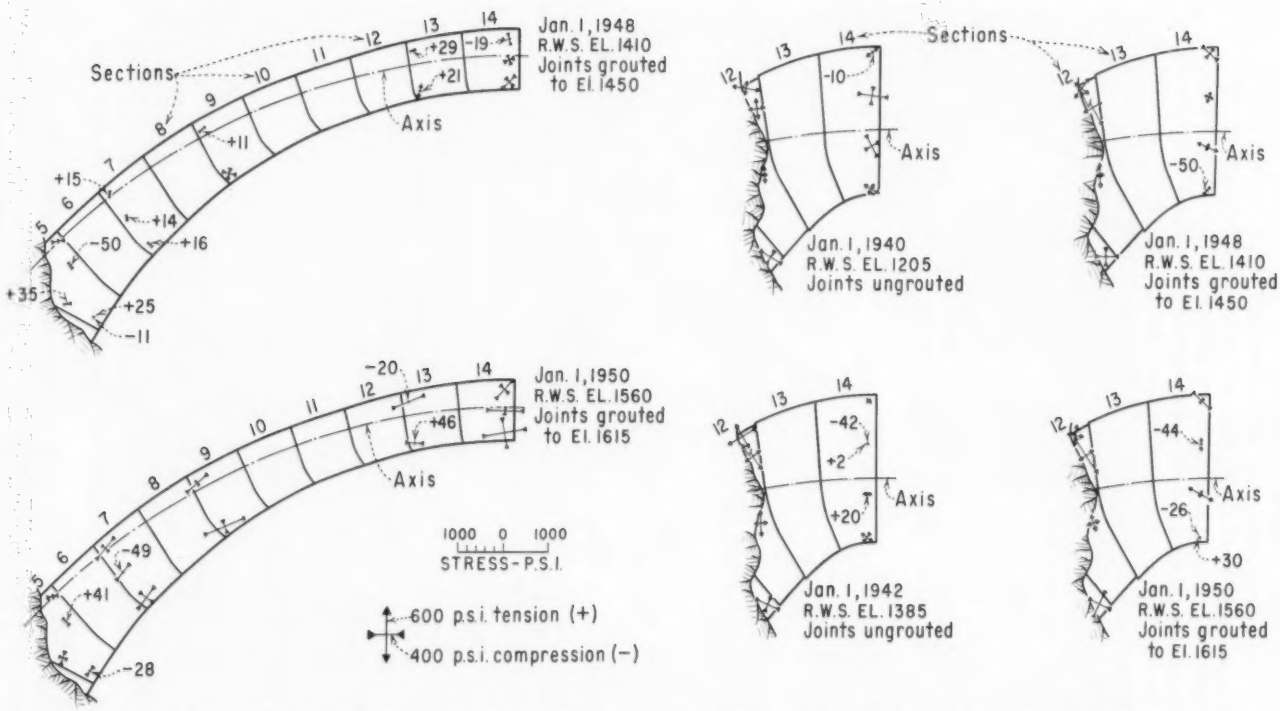


Figure 7. Principal Stresses—Vertical Plane.



PRINCIPAL STRESSES

Figure 8. Principal Stresses—Planes of Elevation 1450 and Elevation 1200 Arches.

a structure during the maximum loading or some other fixed operation condition. The inservice conditions may not be comparable, since loading, as represented in the analytical study, may not occur on the prototype for a number of years after completion of the structure, or may never occur. A principal reason for difference between observed and computed stress appears to be due to the transient temperature changes which are not ordinarily included in an analytical stress study. The stresses due to these changes are often of the same order of magnitude as the load stresses. The effect of these transient temperature changes is one of the most striking features brought out by an investigation of this type. Only through the continuing joint efforts of engineers connected with both measurement and analytical studies of structures, can sufficient information be accumulated that will be useful as a means to effect the narrowing of the gap that exists between design and inservice conditions for a structure. The result will be the assurance that the as-built structure will operate stress-wise as anticipated by the design.

INTRODUCTION

As part of the program of determining the behavior of its structures the Department of Lighting conducted strain relief investigations at both Ross and Diablo Dams. The principal investigation was conducted at Diablo Dam where carefully planned test measurements were performed over a period of approximately 2 months.

Diablo Dam

Diablo Dam shown in Figure 9 is located on the Skagit River approximately 4-1/2 miles downstream from Ross Dam. Diablo Lake establishes the tail-water level for the Ross Powerplant. The dam is a constant angle arched structure with a gravity section at each abutment containing the spillways. The plan and sections of the dam as constructed are shown in Figures 10 and 11 respectively. The dam has a maximum height of 389 feet, a crest length of 1,180 feet and a volume of 350,000 cubic yards of concrete. The arch portion of the dam is 588 feet in length. The thickness at the crown varies from 146 feet at the base to 16 feet at the crest. The spillways are designed to discharge 121,000 cubic feet per second through 19 - 20.5- by 19-foot radial gates.

Construction and Service History

Diablo Dam was designed for the city of Seattle by the Constant Angle Arch Dam Company of San Francisco, California. The design of the dam was based on the assumption that the arch rings carried the entire waterload to the abutments with no transfer of load to the foundation by cantilever or vertical beam action. Contraction joints in the dam were spaced approximately 73 feet apart measured on the centerline of the crest. No provision was made for artificial cooling.

Although the dam was designed on the basic assumption that all of the waterload would be carried by arch action, the contraction joints were not grouted when the dam was completed in 1930. A 3-year period was to be

Figure 9. Diablo Dam and Reservoir.

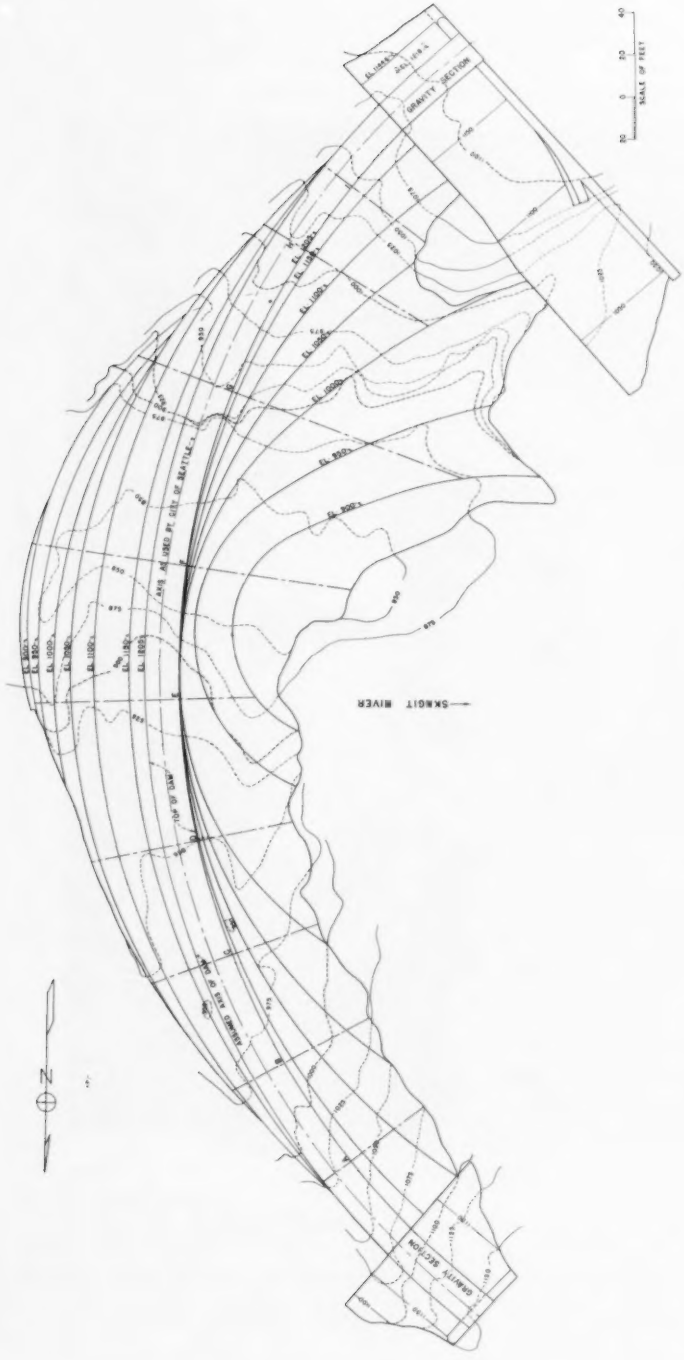


Figure 10. Diablo Dam as Constructed—Plan.

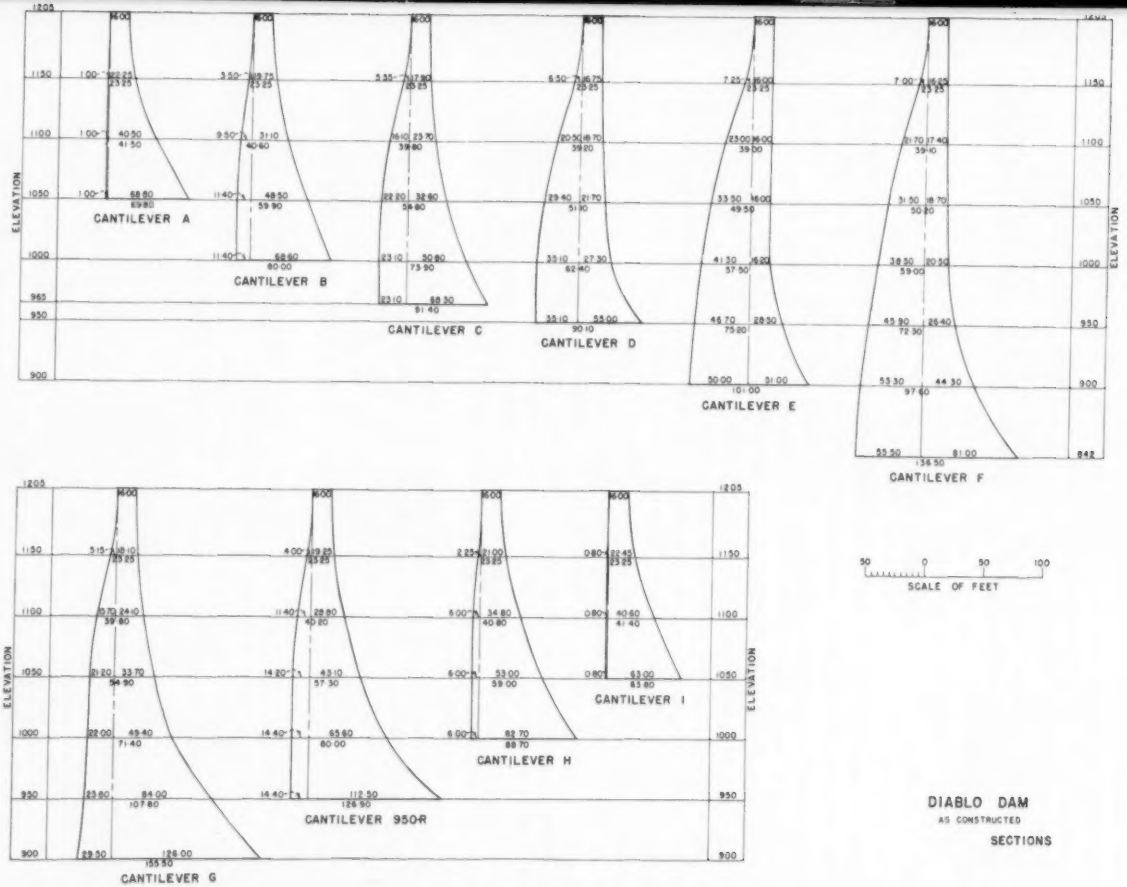


Figure 11. Diablo Dam as Constructed—Sections.

allowed for attainment of thermal stability before grouting the joints. The grouting of the contraction joints did not take place, however, until a program of general rehabilitation and improvements was initiated in 1950. In addition to contraction joint grouting, this work consisted of some foundation grouting, paving areas of exposed rock downstream from both spillways and adding some concrete to the left bank thrust block.

Construction of the dam was begun in October 1927, by the Winston Brothers Construction Company. It was completed in November 1930.

The rehabilitation and improvement work was by Morrison-Knudsen Company in 1950 and 1951.

Procedures

Prior to undertaking the major rehabilitation program for Diablo Dam, a method was sought whereby existing stresses could be evaluated in order to study the dam's structural behavior. The strain relief method was chosen because it had been used successfully to measure residual rock stresses in two tunnels and had been verified in the laboratory.

Using this relaxation method, in which strains are relieved by core drilling around SR-4 gage rosettes attached to the concrete surface, measurements of the existing or residual strain were obtained at 13 locations on the downstream face of Diablo Dam. Measurements were also made at two locations on the downstream face of Ross Dam for comparison. After each test on the surface, the procedure was repeated at the bottom of the core holes thus drilled, 16 inches beneath the surface at these same locations. The tests involved the installation of rock-wool insulating blankets and enclosed work platforms suspended from cables and anchored to the dam for protection from the inclement weather. In addition, strain relief was measured at six surface points on the side walls and top of the sluiceway tunnel through Diablo Dam.

Briefly stated, the strain relief method consists of the following steps: choosing representative locations, preparing the surface, attaching and waterproofing suitable gages, taking strain readings before and after drilling, core drilling to relieve strain, and determining the elastic and creep properties of the material from the cores thus drilled to permit conversion of strain to stress.

After selecting a gage location, an area 10 feet square is covered with a 2-inch-thick rock-wool insulating blanket for at least 48 hours prior to taking strain measurements and this remains in place until the tests are completed. The blanket has a 2-foot-square panel in the center which can be opened to permit access. Through this opening the area on the face of the dam on which the gage will be fastened is chipped and ground smooth. The concrete surface is then dried with heat lamps, thoroughly cleaned, and an SR-4 gage rosette backed with a piece of aluminum foil is cemented in place in the center of a 6-inch-diameter circle marked on the prepared surface. Heat lamps are also used to dry the gage cement. At the end of the drying period the gages are covered with asphalt mastic waterproofing.

With the gage in place and the temperature of the air and the concrete recorded, an electronic bridge is adjusted to balance the resistance in each of the wires of the rosette against the resistance of a dummy gage attached to an unstrained piece of concrete. The gage is then covered with a small

metal pie plate, which is sealed in place to protect the gage from water during the ensuing core drilling operation. Using a 6-inch bit, an annular ring is cut around the gage rosette to a depth of 16 inches. The flow of cooling water to the drill is regulated to maintain an outgoing temperature at or slightly above the initial temperature of the concrete. As a result the initial and final gage readings are taken at approximately the same temperature. When the core is broken out the strain in the concrete is relieved. Another bridge reading is taken and the change in resistance, either positive or negative, is interpreted in terms of microinches of residual strain. This is converted to stress after the elastic and creep properties of the concrete cores are determined in the laboratory. When the first core is removed, the whole process is repeated at the bottom of the hole, i.e., the surface at the bottom is prepared, a rosette is placed, readings taken, and a second core 16 inches long removed from the same hole.

Reasons for Use of an Insulating Blanket

If strain measurements for the determination of stresses are to be made at the surface of a dam, it is essential that fluctuations of temperature be eliminated since temperature changes in the concrete will produce stresses which tend to obscure the stress due to loads. The temperature stabilization needed in this case was obtained by the use of an insulating blanket. It was considered necessary to suppress the daily changes over an area having a width which was large compared to the depth of penetration of the daily temperature changes. In unprotected concrete these changes are felt to a depth of about 3 feet, and a width of 10 feet was therefore chosen as being sufficient for this purpose.

An estimate of the thickness of insulation needed to reduce the temperature changes at the face of the dam to tolerable amounts can be made by the methods of the mathematical theory of the conduction of heat in solids.[6] Such estimates indicate that a 1 inch thickness of the ordinary types of insulation will reduce the amplitude of the daily temperature changes of concrete protected by it, to about 6 percent of the amplitudes which would prevail if it were unprotected. This amount of stabilization would give tolerable conditions for strain measurement.

If the insulation is to be effective, it must be protected from rain, and wind must not be allowed to cause air movement between the insulation and the concrete. To protect against these possibilities, 2-inch nominal semi-thick rock-wool batts with vapor barrier were attached to 3/4-inch plywood panels and then protected by an additional covering of waterproof paper and wire mesh which was wrapped around the edges and fastened on top. This entire assembly was then anchor bolted to the concrete at the point where measurements were to be made. Two 4- by 10-foot sheets of plywood with a 2- by 10-foot piece between were used to provide a 10-foot square panel. A portion of the 2-foot-wide strip was removable at the center to give access to the concrete surface.

Incidental to this investigation, temperature recording instruments were installed at suitable locations to obtain continuous observations during the

6. "Conduction of Heat in Solids," by H. S. Carslaw and J. C. Jaeger, Oxford University Press 1948, Paragraph 24, page 55, Case ii.

course of the field tests. These included concrete surface temperature exposed to the sun, concrete temperature in the shade, and air temperature and humidity in the immediate vicinity of the dam. Reservoir water temperatures were measured at intervals of depth immediately upstream from the dam at the beginning and at the end of the test period.

Subsequent to these observations a joint grouting program was carried through to improve structural action in the dam.

RESULTS

Although the analysis is not yet complete, principal residual strains of the order of magnitude of 130 microinches tension and 120 microinches compression were observed at Diablo Dam. Secant modulus of elasticity and Poisson's ratio at 250 psi stress varied from 2.94 to 6.10 million psi and from 0.01 to 0.18, respectively. Using these elastic constants for converting strain to stress resulted in principal stresses ranging from approximately 450 psi tension to 550 psi compression at Diablo Dam.

Tee-delta rosettes consisting of four gages were used, which furnished one more strain measurement than necessary. This extra gage provided a compatability check on orthogonal strains and was used to compute standard errors by the method of least squares as a means of evaluating accuracy of the data obtained. The average standard error thus computed was $\pm$ 50 psi.

CONCLUSIONS

Although the results of this investigation have not been completely analyzed, it may be stated that the measured values fall within a range considered reasonable for dam structures. The strain relief method has been tested in the laboratory under known stress conditions and found to be reliable. We have complete confidence in the strain measurements obtained by this method and are devoting our present efforts toward a more realistic conversion from strain to stress which will take into account the imperfect elasticity of the concrete.

ACKNOWLEDGMENT

The results presented in this paper represent the combined efforts of many individuals and are by no means entirely the efforts of the authors. Results of the strain-meter investigation represent the combined efforts of teams of workers in the Dams Branch and in the Engineering Laboratories of the Office of the Assistant Commissioner and Chief Engineer, Bureau of Reclamation, Denver, Colorado; and of field and office personnel at Ross Dam and in the office of the City of Seattle Department of Lighting. The efforts of C. R. Hoidal, Designing Engineer, Seattle Department of Lighting, who contributed the description and history of Ross and Diablo Dams, valuable suggestions, and review of the paper, and J. M. Raphael, former Bureau of Reclamation engineer and pioneer in the field of embedded instrument measurements, who so ably directed the strain-meter analysis through to completion and organized the complex computations for processing by punched cards and high-speed digital computors, are gratefully acknowledged.

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ENGINEERING FOR EXPANDING FOREIGN POWER COMPANIES

John F. Pett,[1] A.M. ASCE
(Proc. Paper 1268)

SYNOPSIS

Many interesting problems have been encountered by engineers who have had a hand in the rapid expansion of electric power developments in Latin America. In a companion paper there is discussed the design and construction of Peixoto Hydro-Electric Development, in Brazil, one of the largest power plants built to date in Latin America. This paper discusses special problems that have been experienced by the utilities in those regions, describes a number of specific plants illustrating those problems and finally outlines some of the background and the planning that led to the construction of the Peixoto Plant.

INTRODUCTION

The phenomenal increase in generation of electric energy in the United States during the last two decades is a familiar subject to engineers engaged in the field. There has been a parallel growth in Latin America during the same period, which in some respects is more striking because of its small beginnings and the comparative lack of resources in the various countries involved.

Table I shows the growth of electric energy generation in the quarter century 1929 to 1954, in comparison with that in the United States. The mean annual rate of increase was 7.5% in Latin America versus 6.7% in the United States.

This growth, general in varying degrees in the area from Chile in the South to Mexico in the North has to a great extent been possible because of the financial help of the United States. This assistance has been in the form of investment by private companies and of loans and grants by U. S. Government agencies.

Note: Discussion open until November 1, 1957. Paper 1268 is part of the copyrighted Journal of the Power Division of the American Society of Civil Engineers, Vol. 83, No. PO 3, June, 1957.

1. Cons. Civ. Engr., Ebasco International Corp., New York, N. Y.

The available financial resources for expansion of foreign utility plant from whatever source, have frequently been less than required. The various utilities, both private and governmental, have been in the position of having to get the most possible with the money available without being able to meet completely the demands upon them. This has required a high degree of engineering skill in all the fields of planning, design, construction, and operation.

Historical Background

The largest American company in the electric utility field in Latin America is American & Foreign Power Company, Inc. (AMFORP) which through its associated companies operates in eleven Central and South American countries. The total present installed capacity of its 45 separate operating systems is about 1,350,000 kw. An additional 670,000 kw is now in process of design and construction while almost 1,000,000 kw of additional capacity is in the stage of active planning and investigation.

Other companies and government bodies also have been very active. Noteworthy is the Miguel Aleman System of the Comision Federal de Electricidad of Mexico. The high head plants of Sao Paulo Light and Power Company (Brazil) have been discussed in numerous news articles and technical publications. Centrais Electricas de Minas Gerais (CEMIG) has been actively engaged in construction and planning of hydroelectric plants in Brazil as has Empresa Nacional de Electricidad, S A (ENDESA) in Chile. Other companies and agencies have been engaged in planning and construction to keep generating capacity in other countries on a level with the rapidly increasing power requirements of all of Latin America.

The engineering problems encountered in building and operating these systems have been many and varied. Those discussed herein will be primarily those of the AMFORP system. However, the same or similar problems have been faced by all of the utilities operating in this area.

The original backbone of the AMFORP system was purchased some 30 or more years ago from many separate predecessor companies, which had been operating in some cases since the late 1890's. These plants had been constructed by different groups, with various ideas and standards of design, construction and reliability of operations. For the most part single plants, or small groups of plants had been built to serve separate small load areas. Operating voltages and frequencies varied between adjacent areas. In fact the serious problem of difference in frequency has persisted to the present in a few areas.

The peak load demand of the AMFORP Companies has increased 151% and the annual kwh load 169% during the 15 years 1940 to 1955. This load growth has been met by interconnection of isolated groups where practical, by reconstruction and extension of some plants, redevelopment of some sites, interchange of power with companies serving adjacent areas, but primarily by the construction of new hydro, steam, and diesel electric plants. In the process some old plants have been scrapped because of obsolescence, inefficiency or high cost of operation and maintenance.

It was possible to carry on this expansion program even during the years of World War II, particularly in Brazil where the areas served were

producing raw materials vital to the United States War effort, but the greater part of the new capacity has been installed since 1945.

System Planning

At the time the AMFORP companies were organized it was evident that substantial capacity expansion would be required before many years. It was natural that in many areas attention would be directed to the relatively untouched hydro resources.

Investigation of potential hydro projects was handicapped originally by lack of basic geographic and hydrologic knowledge. Only small areas had been accurately surveyed and mapped. Rainfall and streamflow data were nearly nonexistent.

One of the first steps in planning was a program of preliminary reconnaissance of many rivers and hydro power sites. This investigation was far-reaching, covering all of the countries and most of the areas in which the companies were operating. It was determined that some sites were worthless, other merited immediate further consideration, while others should be reserved for intensive study in the more distant future. With few exceptions it will be found that rivers with hydro possibilities and within reasonable transmission distance of load areas have been subject to study by engineers representing various interests. Some of these studies started 40 or more years ago even on rivers in remote areas.

Since World War II the quality and scope of mapping of the undeveloped areas of Latin America has greatly improved. This is due first to the technical advancement in aerial photographic mapping and secondly to the need for good maps for many purposes in these regions of rapidly expanding agricultural, industrial, and transportation development.

In some cases this mapping has been done as part of comprehensive government programs. But in numerous instances it has been necessary for the utilities to have aerial surveys made of specific areas in which they are interested. Some of these surveys have covered hundreds of square miles. Usually the work has been contracted to companies specializing in the field, except that the ground control work has been carried out by the engineers of the utility company.

In most of the countries there is not even now a systematic comprehensive program of streamgauging. The various companies initiated such programs on many rivers with apparent hydro potentialities many years ago. In Brazil particularly, many good gauging stations were established in the late 1920's. Daily records of flow at many of these stations have been recorded since then. During the first few years hundreds of current meter measurements were made to establish station rating curves. Subsequently sufficient check measurements have been made to detect any changes in the flow characteristics at the gauging section. But the principal cause of error in reported data is carelessness of the gauge readers who in remote areas necessarily have been untrained local people hired simply to read the staff guage twice a day and submit periodic reports. When two or more guages exist on a single river or adjacent drainage areas it is not difficult to detect that a guage reader is careless, or perhaps using his imagination to avoid a long hike to the gauging station.

Usually it has been difficult to obtain the equipment and trained personnel required for extensive investigation of foundation conditions for dams and other important structures prior to starting of construction. However, there are becoming increasingly available organizations specializing in this field with suitable equipment and personnel, especially in regions where mining is important or when there has been considerable recent construction activity. On a number of recent jobs the seismic method has been used to supplement data obtained by core drilling and test pits. It is hoped that as experience is gained it will be possible to reduce the amount of drilling without sacrifice in quality of data obtained. This will result in very considerable reduction in cost of investigation.

The development of air transportation in Latin America has been a great boon. Few areas cannot be reached comfortably now within a day or two from major U. S. cities, in contrast to only a few years ago when weeks of travel and considerable discomfort were often involved.

It will be noted from Table I that hydro generation in Latin America has increased at nearly twice the rate as thermal generation. But there are still many hydro sites yet to be developed. In each case where a potential hydro site in Latin America is under consideration there are special factors which must be taken into account. Some of these are:

1) Availability in the country of oil or coal for operation of an alternate thermal installation.
2) Financial and economic conditions in the country which would limit or prevent expenditures for equipment and materials that must be imported.

As previously mentioned many hydro sites when first studied were not adaptable to development at reasonable cost for the amount of capacity required at that time. However, due to increase in magnitude of load and expansion of areas served, such projects must be re-examined periodically in the light of changing conditions.

A major factor in re-evaluation of hydro projects is the addition of appreciable thermal capacity to a previously all-hydro system. For example, a number of hydro projects which were carefully investigated some years ago as sources of added capacity for one hydro system were discarded or postponed because of high cost per kw of low water season capability. In each case there was a wide range between high and low water season streamflow and storage possibilities were limited. In addition the annual system peak load occurred in the dry season. There was no market for hydro capacity in excess of low water capacity. What the system required was reliable low water season peaking capability. But now an appreciable block of thermal capacity has been added. The economic advantages of the previously expensive hydro sites is now being re-evaluated to take into account the usability of what was formerly unsaleable dump wet season energy which can now be used to reduce high cost thermal output.

Another factor affecting the re-evaluation of sites is the increasing tendency for interconnection of previously isolated systems. Until recently most utilities in Latin America have served areas relatively remote from one another. The magnitude of loads was such that major transmission connections were not economical or practical. However, geographic expansion of

service areas and load growth have reached the stage when such interconnections are becoming increasingly feasible. Thus again hydro projects which were too large and costly for a single small system, or too remote, or perhaps subject to appreciable seasonal variation in output are now in a more competitive position than when originally proposed.

Redevelopment of Projects

On the whole the old plants were well conceived and constructed. Many have given reliable service for 40 years or more without undue maintenance and can be expected to continue for years to come. However, there are some which for one reason or another have offered special problems.

A number of hydro developments utilized only a small part of the total potential of the site. Figure 1 shows the Americana Hydro-Electric Development in the State of Sao Paulo, Brazil. This plant, with an installation of 3 - 10,000-kw units utilizing a head of 107 ft. was put in service in 1949 to replace an old plant with an installation of 2,180 kw and utilizing only 51 ft. of the available head.

Similarly the Avanhandava Hydro-Electric Plant also in the State of Sao Paulo replaced a 1,900-kw 26 ft. head plant with a 30,000-kw plant developing a total head of 59 ft.

At Americana the large increase of capability is possible because of the added head, the provision of 120 million cubic feet of storage, and because the plant can be operated at much lower load factor as part of a large interconnected system than when it was serving a small isolated load.

At Avanhandava the capacity increase was possible because of the greater head utilized and full utilization of available flow.

There are still a number of similar situations where ultimately the site will be redeveloped for more efficient use of the available potential. But it should be borne in mind that when the original plants were built, the amount of capacity required was only a small part of that ultimately possible. It was economics rather than lack of foresight that dictated the initial partial developments.

Thermal Installations

In some countries hydro resources are extremely limited. Expansion of capability in these areas has largely been through gradual interconnection of isolated systems, expansion of existing plants and construction of new modern steam electric stations. The recent trend has been to oil fired stations because of scarcity of good coal in most of these countries.

The size of individual units has been increasing steadily. At present three units of 60-mw rated capacity are being installed in Cuba. These are the largest units that can be conveniently transported to the sites at this time.

Design Problems

As previously stated the design of the older purchased plants was done by engineers with many different ideas and standards of design.

In a number of such plants there was not sufficient appreciation by the original designers of the operating difficulties that would be experienced due to the transportation of tremendous volumes of sand, gravel, boulders, and other debris during the flood season. When these plants were constructed some 30 or more years ago the drainage areas upstream from the point of diversion were largely uninhabited. Gradual settlement of these areas with resultant deforestation, road construction, and cultivation of the land caused increasingly greater floods and severe erosion.

Figure 3 shows the type of material carried by floods in a steep valley in the Andes of Colombia. Even the largest boulders shown have been transported to their present location by recent floods. The normal flow of this river is about 100 to 150 cfs.

In several cases intake facilities formerly entirely adequate are now practically inoperable, and ultimately must be reconstructed and replaced. Mechanization of intake and sluice gates that were originally manually operated has helped when conditions are not too serious. Small silting basins have been added at some plants where the flood-borne material is principally sand and silt. This has resulted in marked increase in the life of turbine parts subject to erosion.

This problem has been handled with reasonable success in the design of recent plants. Figure 2 shows the dam and intake of the Volcan Hydro-Electric Plant in Chile. This plant, put into service in 1944, has an installation of 13,000 kw, using 320 cfs through a net operating head of 575 ft. The flow line from the intake to the forebay consists of 40,500 ft. of canal and tunnel. Near the intake is a double chamber sand settling basin with a volume of 975,000 cu. ft. which has been very effective in removing sand and silt which passes through the intake. Particular consideration was given in design of the dam and intake to the problem of flood-borne debris. It can be seen in the photograph that the sill of the taintor gates is located about at the level of the original river bed. At time of flood the 19.7 ft. x 13.1 ft. high taintor gates can be raised leaving a practically unobstructed flood channel.

Figure 4 shows the general layout and pertinent details of the dam and intake of the Nuestro Amo Hydro-Electric Development near San Jose, Costa Rica. This structure was also designed to eliminate as much as possible of the sand and gravel carried by Rio Segundo in the flood season. Its essential features are:

1) A sluiceway with Gate A at its upstream end, in front of the intake screens to pass downstream as much bed load material as possible. Intermittent opening of Gate A for short periods makes it possible to wash out any material which tends to settle out and pile up in front of the intake screens.
2) A skimming weir B over which water must pass at low velocity to enter the canal leading to the storage reservoir.
3) A sluice Gate C for washing out of sediment deposited in the entrance canal in front of the weir.

The Volcan intake has substantially the same arrangement of canals, gates, and weir as Nuestro Amo. At some old plants it has been possible to modify existing structures so as to permit operation on somewhat the same principle, that is, (1) to make it possible to pass on downstream through the dam without obstruction, the greater part of the debris, particularly timbers,

boulders, gravel and the like and (2) to remove whatever sand and silt that enters through the intake as soon as possible before it passes into the main canal. It is important that it be possible to open and close the sluice and intake gates rapidly in order to wash out the sand, gravel, etc. as it is deposited at various points.

In these cases adequate intake facilities were provided at the time of initial construction. Studies are now in progress on the practicability of reconstructing the intakes of several of the older plants according to the same principle.

An essential consideration in studying methods of improvement or reconstruction of portions of old plants is the amount of money that can justifiably be spent on a plant that is approaching obsolescence. Without doubt in rapidly growing systems there are many small plants which will be shut down during the next five to ten years. But in isolated smaller systems, similar small plants must be kept in service for many years and it is for these plants that economical methods must be found of improving reliability and lowering the cost of operation and maintenance.

Cavitation

Another factor that has caused trouble at some of the very old plants is the very high setting of turbine above tail water. This has resulted in excessive cavitation. Where this has occurred in conjunction with a seasonal heavy sand load due to poor intake facilities, real trouble has been experienced.

At a few plants it has been necessary to construct a low weir in the tailrace to keep the draft head within a reasonably acceptable limit. At one plant where a third unit was installed recently it was necessary to install the new unit at a lower elevation than the old units in spite of the disadvantage of change in operating floor level.

Another occasional practice of the designers of the older plants was omission of surge tanks. Originally this was possible because the gate closure time could be made very long. With small isolated plants (all hydro) serving mostly non-industrial loads, even very considerable speed variation was not particularly objectionable. However, as steam turbines with inherently closer speed control requirements have been added to the system, the problem of speed control has become of increasing importance.

Recent Projects

The Guaricana Hydro-Electric Plant near Curitiba, Brazil is an example of recent design for a small station. This plant is expected to go into commercial operation early in 1957 with an initial installation of two 7,500-kw units, operating with a net head of 1,030 ft. Provision is made for an ultimate total installation of 40,000-kw.

The general project arrangement is shown in Figure 5.

The principal feature of the project is the 7,000 ft. upper level tunnel from intake to surge tank, a 1,350 ft. long inclined shaft and 985 ft. long penstock gallery. Only about 800 ft. of the upper funnel is lined. The inclined shaft is circular, lined with unreinforced concrete. The rock formation is generally very sound gneiss and only an insignificant amount of seepage appeared in the

shaft during construction. The shaft will be heavily grouted after the lining is placed. Grouting pressure at any point will be 10% in excess of the maximum dynamic head.

There is precedent for the simple concrete lining of the high pressure shaft, at the nearby Chamine Hydro-Electric Plant and elsewhere. At Chamine built by the same company in 1929, there is a similar inclined shaft in poorer rock under a maximum head of 780 ft. which has been entirely adequate.

At Guaricana the surge chamber, excavated within the rock, is of the restricted orifice type.

In planning the Guaricana layout, comparisons were made of alternate conduit schemes, including an outside surface penstock from surge tank to powerhouse, vertical shaft instead of inclined shaft, and also underground powerhouse, arrangements with the power house located either near the intake or just inside the mountain near the downstream end of the gallery, or at the foot of the inclined shaft.

The underground power house schemes had the disadvantage that the power house must be excavated for the ultimate 40,000-kw installation although only 15,000 kw was being installed initially.

The possibility of slides damaging a surface penstock originally lead to consideration of the underground shaft schemes. Further field investigation showed that the slide hazard probably would be small. However, cost estimates of the alternate layouts lead to the conclusion that the most advantageous arrangement would be that shown in Figure 5.

Construction and maintenance of access roads to and around the job was a major undertaking. The principal construction obstacle at Guaricana was the almost continuous rainfall. The terrain is rugged and covered with thick jungle growth.

It is reported that during the early days of access road reconnaisance, a survey party spent 30 arduous days travelling a distance of 15 miles cross-country without seeing the sun once during the period due to the thick vegetation.

Because of the possibility of a shortage of cement and the cost of hauling it to the job site, concrete arch and composite rock-earthfill dam designs were considered as alternate to the gravity dam which was finally adopted on the basis of lowest cost. Poor abutment rock conditions were unfavorable to the arch design. Lack of suitable materials close to the damsite and unfavorable topography were disadvantageous for the rock-earthfill type. The incessant rainfall was another disadvantage as it would have been difficult to control the moisture content of the impervious fill.

The Regla Steam-Electric Station near Havana, Cuba is shown in Figure 6. Construction was started in April 1952, the first 30,000-kw unit went in service in February 1954, unit #2 rated at 40,000-kw capacity in August 1955 and unit #3 rated at 60,000 kw is now in construction. Unit #4 also 60,000 kw will follow shortly. Foundation conditions were investigated very thoroughly by means of drill holes, test pits and pile loading tests. Steel tube piles were used for underpinning units 1, 3 and 4, but under parts of unit #2 structures, rock was at comparatively shallow depth and 48 in. to 66 in. diameter caissons were adopted as a cheaper and better solution.

Availability of local materials and restriction on importation of such materials as structural steel are factors that frequently affect choice of

designs. Figure 7 shows the architectural treatment of extensions to two plants in Mexico which are now being designed. In each case two 15,000-kw units are being added to existing plants. The design of the two extensions will be essentially duplicates except for foundations. The present station buildings are steel frame with corrugated asbestos roofing and siding. The new extensions will be reinforced concrete rigid frames, with brick walls and concrete roof slabs.

One reason for substitution of reinforced concrete in place of structural steel was the possibility that the supply of steel in Mexico might be overtaxed by an intensive construction program including simultaneous extensions of 7 existing stations. Also the concrete frame lent itself very well to the desired architectural treatment of the new extensions. It will be noted in Figure 7 that the unattractive old structure is pretty well obscured by the new construction with only a minimum of changes to the old building.

Material availability has also dictated the type of pile used in a number of plants. Reinforced concrete piles have frequently been used because it was impossible to import steel piles.

Several companies have developed very efficient processes for local manufacture of reinforced concrete poles for distribution circuits and similar purposes, because of the high cost and restrictions on the importation of the more commonly used wood and steel poles.

Engineering and design for the construction carried out for the AMFORP Companies has generally been the responsibility of two associated companies, Ebasco International Corporation and Ebasco Services Incorporated, in cooperation with the local company. Occasionally it has been more advantageous for the local company to handle its own design. Also on several recent steam plant installations "package plants" have been purchased, that is, the manufacturer of the steam turbine generator has contracted to design and furnish all materials for the plant except the foundations, and of course general site improvement. This procedure has been followed more often in the case of substations.

Construction has been carried out generally by the construction organization of the individual operating company although it is not unusual in certain areas to contract all or part of a job to local construction organizations.

Very close cooperation and coordination of effort between the field engineering and construction organization and the New York engineering and design staff is necessary in order to avoid construction delays and errors. Lack of easy direct and rapid communication between the design office and the construction site aggravates this problem. The time required for transportation of equipment and materials from the United States or Europe to the job necessitates more than the normal advance planning. It is not unusual for two or three months to be consumed in the transportation of equipment from the manufacturer's plant to the job.

Construction drawings are prepared customarily in the metric system which is the standard in the various countries. However, structural steel drawings and some others where the material is to be fabricated in the United States are dimensioned in feet and inches with the additional feature that principal dimensions such as column spacing are also shown in millimeters where this will be of assistance to the construction organizations.

Effort is made in planning new extensions to duplicate insofar as possible the design of previously built stations. This is in order to reduce design

time and costs. However, it has been found that even where fairly close duplication of mechanical and electrical layout is possible, the enclosing building structure and foundations do not tend to duplicate too closely, due to differences in foundation conditions and architectural variations due to climatic conditions, local preferences as to materials, or material availability.

Investigation of Peixoto Project

The largest plant built to date by any of the AMFORP companies is the Peixoto hydroelectric project on Rio Grande in Brazil. Its design and construction is discussed in a paper entitled "Peixoto Hydro-Electric Development" by R. Olafsen M. ASCE. It may be of interest to know something of the planning of system development that preceded the selection of Peixoto for construction at this time.

The development is being built by and for Companhia Paulista de Forca e Luz. This Company provides electric service in the north central portion of the State of Sao Paulo. Its transmission lines extend over an area roughly 290 miles long by 220 miles wide.

In 1939 there were 21 generating plants, all hydroelectric, ranging in capacity from 100 to 7,800 kw with a total maximum capability of approximately 44,000 kw. It was divided into three separate not connected groups, two operating at 60 cycles, one at 50 cycles. The load of each sub-system was rapidly approaching generating capability. Studies were carried out to establish both immediate and long-term programs of system capacity additions consistent with forecast load requirements.

Some increase in overall capability was obtained by changing the one area from 50 to 60 cycle operation, interconnection of the three groups of plants, and by rehabilitation of some of the generating equipment. But the first appreciable capacity addition was the construction of the Avanhandava Hydro-Electric Station with an initial installation of 20,000 kw in 1946 followed by a third 10,000-kw unit in 1949.

The next step was the construction of Americana Hydro-Electric Plant with an initial installation of two 10,000-kw units in 1949. Both these plants replaced older small plants, as previously discussed in this paper.

The annual rate of load growth was such that it was obvious a much larger source of new capacity was required. During 1947 and 1948 many new sites within the area served were studied as well as the possibility of further reconstruction of some existing plants. It was concluded that none of these sites was large enough to meet load requirements much beyond the date when it could be put in service. It was then decided to concentrate on the possibilities of Rio Grande, a large river flowing in a generally westerly direction and forming the northern boundary of the State of Sao Paulo and also of the company's service area.

An interim program was inaugurated to take care of load growth until a major new plant could be completed. This interim program included the addition of two hydro units totaling 16,000 kw at two existing stations, plus the construction of the system's first steam plant, Carioba, with an initial installation of two 15,000 kw units. This added capacity went into service in 1954, and brought system installed capacity to approximately 150,000, approximately 3.5 times the installed capacity at the time the expansion program was started 15 years earlier.

There was already one plant, Maribondo, on the lower Rio Grande with a capacity of 9,600 kw. It was known that complete utilization of the Maribondo site would yield in excess of 200,000 kw. There were other promising sites upstream. Company engineers had explored sections of the river as early as 1929. Other engineers had travelled along the river even earlier. Stream-gauging stations were established along the river in 1928 and 1929. Four or five good project sites were known to exist but all were of much greater capacity than had heretofore been required. The problem now was which project should be developed first.

Starting in 1950 the company started a program of further reconnaissance, aerial surveys, detailed ground surveys of special areas, preliminary drilling, and hydrologic investigation. The purpose was to obtain sufficient data for preparation of preliminary project layouts, together with corresponding estimates of cost and project capability. When completed these studies showed that three alternate projects stood out, each with certain advantages. Of the three Peixoto, the farthest upstream, would provide the greatest amount of initial system capacity, at a lower cost per kw, and in addition it would provide some 1.8 million acre feet of usable storage which would increase the capability of any of the alternate plants subsequently built downstream.

It was decided that Peixoto would be built and engineering was authorized and started in August 1951. Hydrological and cost studies indicated that the ultimate project capacity should be in the range of 400,000 kw, equal to 250% increase in Paulista System Capacity.

The size of unit was selected after careful study of transportation limitations. A detailed survey was made of all transportation routes from the port or entry, Santos, to the job site, a distance of approximately 275 miles. This survey covered alternate railway and highway routes, bridge capacities, bridge and tunnel clearances and all other limiting factors. It was determined that the physical dimensions and weights of the components of a 40,000-kw unit were about the maximum that could be handled conveniently at the time.

In comparison with the previous largest units in the system, 15,000 kw, the 40,000-kw unit proposed for Peixoto was a big jump. However, this capacity matched well with the expected rate of load growth. Also it was desirable to make the units as large as possible to cut down the number of units required for the 400,000-kw installation. Therefore 40,000 kw per unit was agreed upon.

Design and construction of Peixoto has gone ahead and the initial two units are expected to go in service early in 1957.

Since the project was originally authorized, it has developed that its total capacity can be utilized sooner than originally contemplated. Recent forecasts indicate a faster rate of load growth. Also it is planned to use a portion of its output to supply an associated company in the State of Minas Gerais. Also there probably will be transmission interconnections with other utilities serving adjacent areas.

During the past year engineering studies have been actually under way to determine what should be the next project constructed and to obtain all of the field data required to proceed with its design and construction at the proper time.

TABLE I

Total Annual Generation of Electric Energy

Billion Kwh

Year	United States	All Latin America 1/		
		Hydro	Other	Total
1929	92.2	2.2	3.5	5.7
1937	118.9	5.2	4.7	9.9
1951	370.7	13.5	11.8	25.3
1954	471.7	-	-	34.8
Mean Annual % Increase 1929-1954	6.7	-	-	7.5

1/ from "Ingenieria Internacional Industria" 1953 Edition

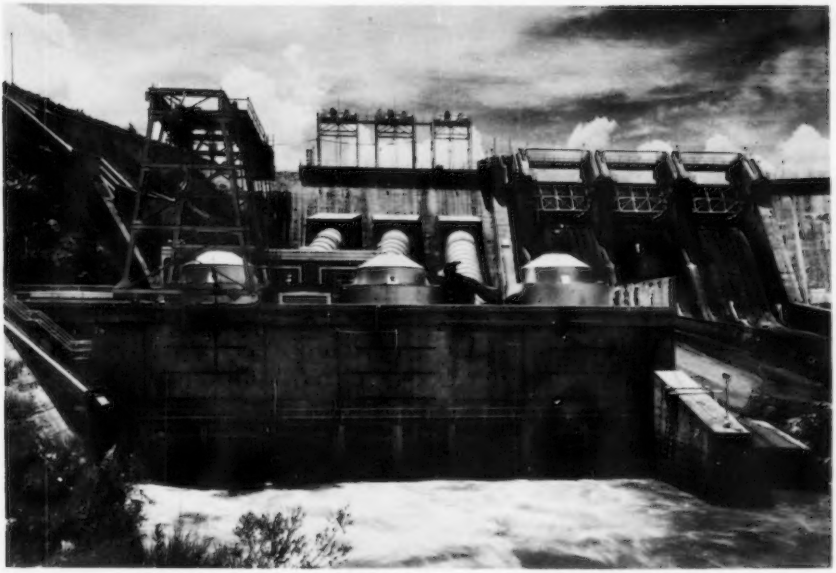
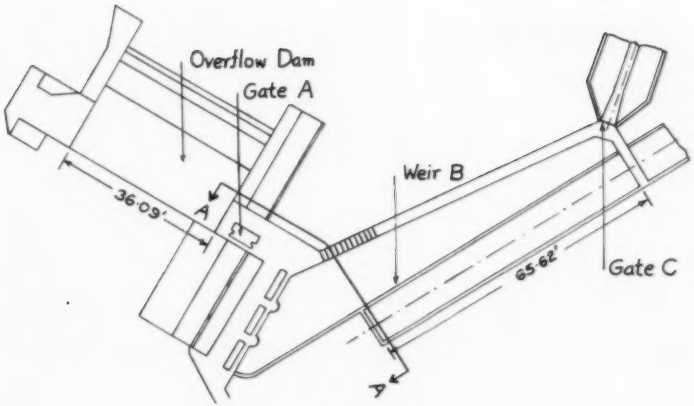


Fig. 1. Americana Hydro-Electric Development.
Brazil

Fig. 2. Dam and Intake. Volcan H. E. Development.
Chile

Fig. 3. Flood-Borne Debris.
Colombia



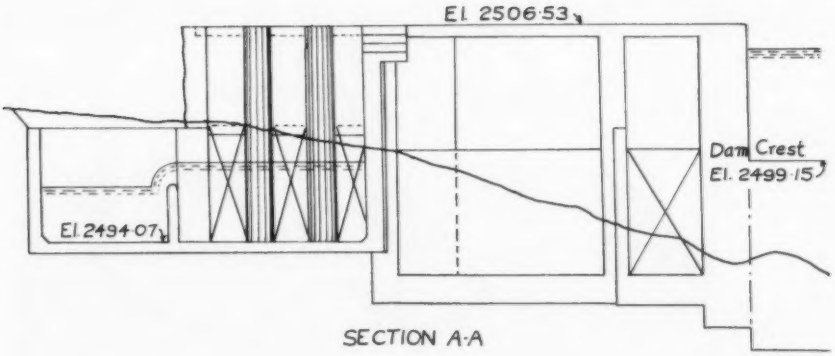


FIG. 4
DAM AND INTAKE
NUESTRO AMO H.E. DEVELOPMENT
COSTA RICA

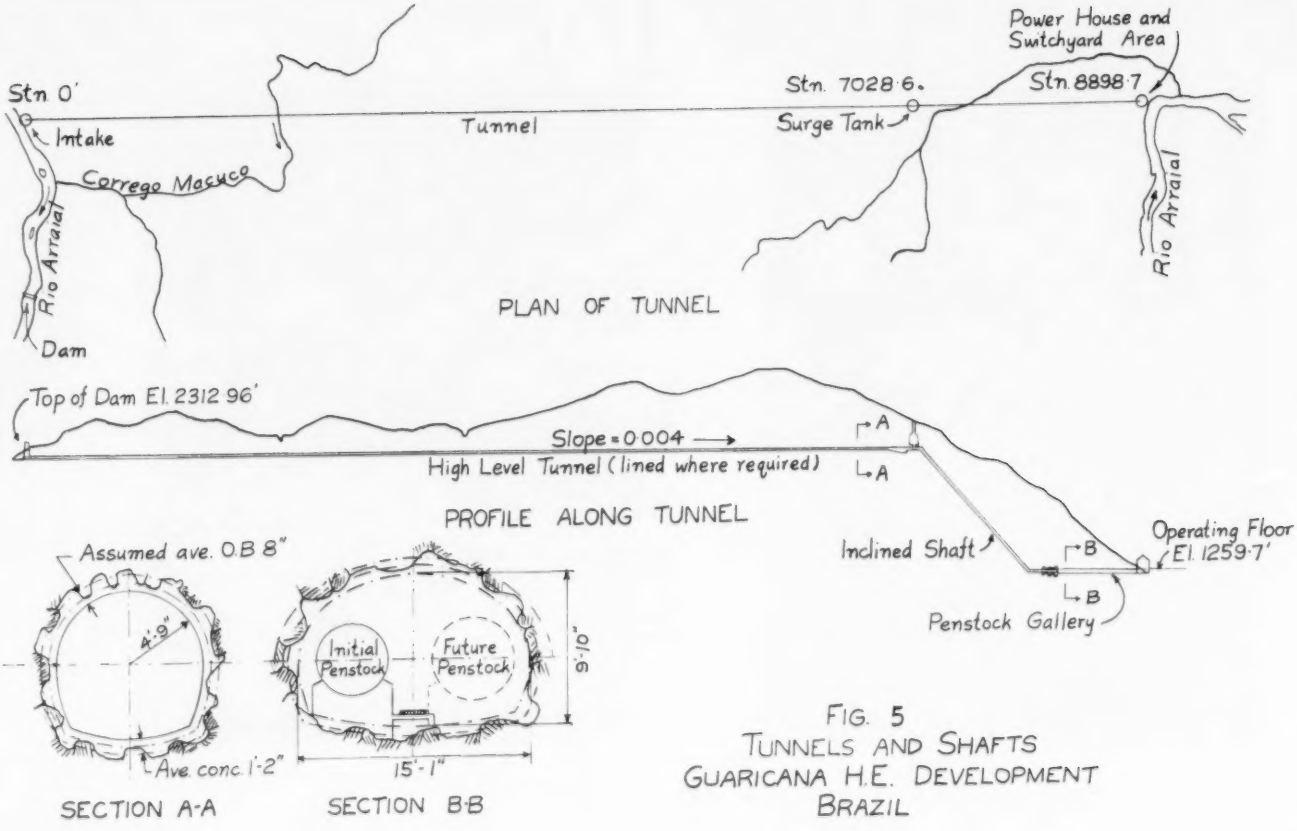


FIG. 5
TUNNELS AND SHAFTS
GUARICANA H.E. DEVELOPMENT
BRAZIL

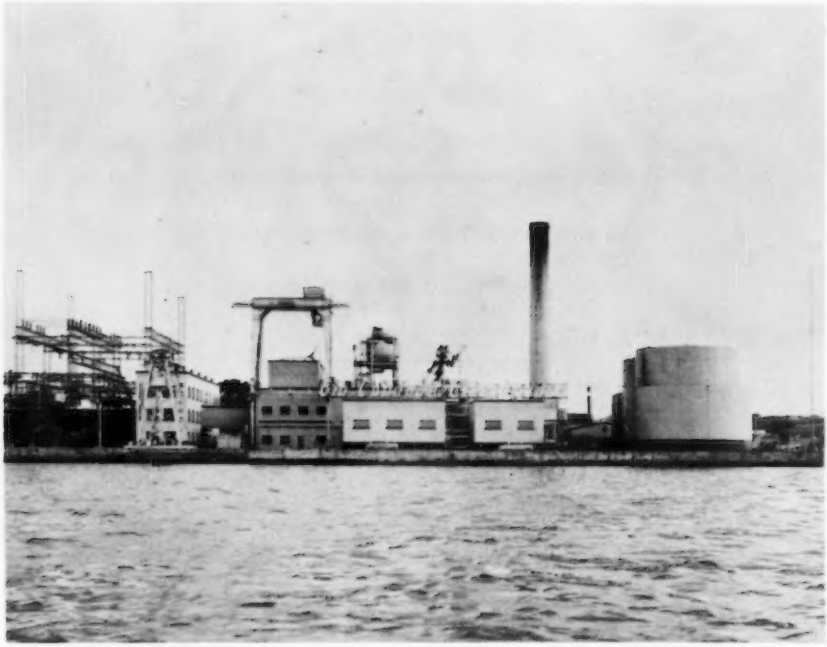


Fig. 6. Regla Steam Electric Station.
Near Havana Cuba

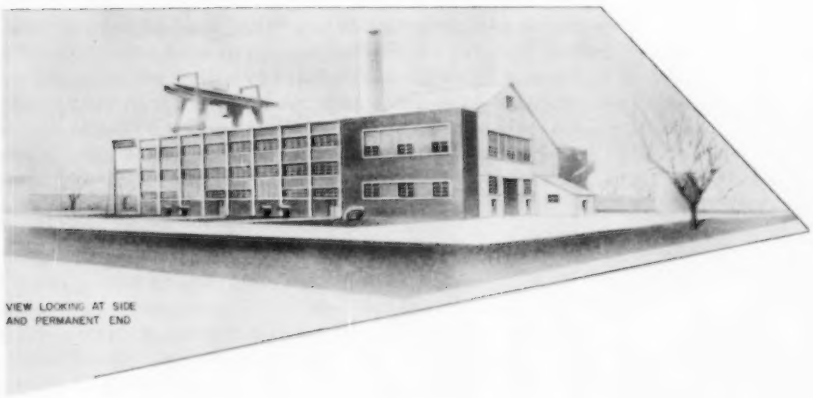


Fig. 7. Proposed Extension of Two Steam Electric Stations.
Mexico

Journal of the
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Proceedings of the American Society of Civil Engineers

SOME PROBLEMS OF A PENSTOCK BUILDER

John N. Pirok,* M. ASCE
(Proc. Paper 1284)

ABSTRACT

The many problems encountered in the building of a penstock are described. These occur in the many phases of the operation, namely, engineering personnel, procurement, fabrication, transportation, erection, welding and testing. Particular attention is paid to those problems encountered during engineering and erection, and their solutions are discussed.

Building a penstock requires the solution of many problems. Some of these problems are simple; others are complex. There are several phases in the operation namely: engineering, personnel, procurement, fabrication, transportation, erection, welding, testing and several others. Each of these subjects could be developed into a separate paper. The major portion of this paper will concern the engineering and erection phases of the operation.

Most penstocks built today are of fusion welded construction. In view of this trend all remarks in this paper will refer to all welded construction. It was suggested the writer cover the operations of a penstock builder following the award of a contract; but operations actually begin when requests for bids are received. The paper, therefore, begins with preparation of the bid.

As anyone would expect, requirements stipulated in bid plans and specifications are quite varied. Some requests for tenders require the bidder to design the complete penstock based on hydraulic gradient information contained in the plans. Other requests for bids have complete designs with intricate details spelling out precisely how the work shall be performed.

Development of hydraulic data is the "baliwick" of the consulting engineer and represents months, sometimes years of work in its preparation. The penstock builder accepts this information without question.

Even though a penstock design may be completely worked out, it is our practice to check the structure to satisfy ourselves nothing was overlooked.

Note: Discussion open until November 1, 1957. Paper 1284 is part of the copyrighted Journal of the Power Division of the American Society of Civil Engineers, Vol. 83 No. PO 3, June, 1957.

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The next step is to make a detailed analysis of all materials specified and determine availability of each type. Materials have been specified that are not readily available. Acceptable alternates must be found and offered in the proposal. Special castings and heat treated alloys require development of reliable sources of supply.

Plans and specifications are scrutinized for special performance procedures such as stress relief, radiographic examination of welding, fit-up tolerances, roundness tolerances, etc. These items though sometimes innocent in appearance materially affect the prices bid for the work. Sometimes allowances are made for special treatment on important structural elements even when not specified, if in our judgment, special treatment is required. We may deem it advisable certain portions of the work be radiographed to assure sound construction. Also thermal stress relief of extremely heavy weldments may be performed several times during the welding operation to avoid weld cracks and distortions. This can account for differences in tenders submitted by various builders.

Fabrication problems are studied in the light of the builder's shop practices and available fabrication facilities. Plans may specify details that cannot be performed by usual fabrication practices. Shapes and forms may require hand forging to comply with the drawings. Rolled wide flange beam sections cannot be cold formed into small curvatures without crippling the webs. Angle sections exceeding roll capacities must be "bull-dozed" into shape. Acceptable built-up sections must be designed to meet specification requirements. Castings specified can sometimes be replaced with weldments at considerable savings in cost.

Methods of forming plate materials and edge preparation for welding are tentatively decided upon. Specified joint details and welding are compared with our standard welding practices. If details are found to be awkward or difficult to perform exceptions are voiced when bids are submitted. Shipping clearances and field erection method determine how much assembly can be performed in the shop.

Erection methods are next considered. Several feasible erection schemes are devised. Each project is different and erection schemes are equally as varied. Good, accurate, site plans assist the penstock builder in devising economical erection procedures. Plan should show topography, roadways, bridges, assembly sites, possible obstacles, headroom, existing handling facilities and access to work areas. Plans and specifications should clearly stipulate who shall provide additional access roads, temporary bridges, etc. or who shall clear work areas. All of these influence the erection scheme to be employed.

Manhour studies must be made for transporting, handling, erecting and welding. Before any method is selected, the construction site must be visited, studied and feasibility of proposed schemes verified. Sometimes none of the schemes are workable because site conditions were materially modified after the bid plans were prepared.

Transportation must next be considered to determine how and in what form fabricated materials can be shipped via common carriers from the shop to the job location; how materials can be transported from common carriers to the preassembly site; and finally how preassemblies will be moved to final location. This phase determines the type of field erection equipment that must be employed.

After considering all the above items, we are ready to submit a bid. This list of items becomes all the more important when work must be performed in remote inaccessible regions.

A great amount of work is done by the bidder up to this point even though he has no contract. A good penstock bid requires adequate time to properly study all items that affect the bid price. In some cases insufficient time is allowed by the party requesting bids. Inadequate time allowance results in guessing at important items, misinterpretation of engineering intents and sometimes bickering over progress after the work is under way. If possible, requests for tenders should be made in the early stages of the project, well in advance of actual construction, with ample time allowed for estimating the job. Such procedure permits orderly planning and execution of the work that will be reflected in the prices quoted.

As soon as a contract is awarded all of the items considered in preparing the bid are implemented into positive actions. The first step is to prepare a bill of materials based on the bid plans and place orders with producers and suppliers. Final bills of materials are prepared after all engineering features are agreed upon and fabrication and erection procedures have been finally established. Prompt placement of orders is very important today because of the tight material situation.

Before any final fabrication or erection plans are started all engineering items are fully developed and cleared with the consulting engineer. Bid plans vary extensively. Some are simple line drawings that show alignment of the penstock, profile of the proposed ground line relative to the penstock and the hydraulic gradient. Some bid plans go into intricate details which often do not fit the fabrication or erection practices of all builders. This necessitates requests for revisions that can result in delays.

Some functions demand unique details. These should be clearly stipulated in the plans and specifications and the details followed explicitly. Such details are sealing rings, closure seams, closure temperatures, location of expansion joints, waterway clearances, etc. The choice of plate width must be left to the discretion of builder to suit his fabrication facilities and the erection scheme he will employ. The exception might be where structural requirements demand specified widths such as thickened rings at pipe supports.

Design, fabrication and erection skills in plant work are constantly being improved. These should be incorporated into each project built. The builder should be encouraged to exercise his ingenuity to bring out new developments. Alternates proposed should be backed by sound engineering and supported with calculations.

Special fittings such as wyes, tees and other odd shaped sections always present fabrication, erection and welding problems to the builder. The problems can be minimized by good layout, proper choice of details, proper welding procedure and in special cases stress relief. The builder needs some latitude here if he is expected to turn out an acceptable job. Some details look good on paper but cannot be practically executed. The erection scheme and shipping clearances determine how much shop assembly can be performed. Economical car loading often dictates whether shipment will be on knocked down or built up basis.

The choice of erection method is dictated by site conditions and on the type of penstock to be built. There are four distinct types of penstocks:

A - Exposed penstocks
B - Penstocks in tunnels
C - Penstocks encased in concrete dams
D - Penstocks buried in the ground

Type A

Exposed penstocks may be on ring supports, steel or concrete cradles, and in rare cases on gravel cradles. Exposed work lends itself to use of crawler cranes where the terrain is mild and an access roadway exists or a temporary roadway can be economically constructed along the tube alignment. Such a roadway permits transporting large assembled sections to the point of erection.

Figure No. 1 - shows an 18 foot diameter ring supported penstock recently completed for the Niagara Mohawk Power Company at their Rainbow Falls Development near South Colton, New York. This penstock is practically horizontal and easily reached with crawler cranes. The terrain permitted construction of access roadways on both sides. One side was used for temporary storage of preassemblies. The other served the crane. Large sections weighing 14 tons each were preassembled and welded in a nearby yard.

Figure No. 2 - shows a ring support assembly on the same project being moved into position.

Figure No. 3 - shows the expansion joint being placed into the line.

This was an ideal erection site.

Figure No. 4 - shows concrete cradles in a rock cut for a penstock built in 1937 in Newfoundland. Again good access was provided by a parallel roadway permitting handling of large preassembled sections with crawler cranes. The cradles were spaced about 8' apart which simplified the support problem.

Figure No. 5 - shows a ring supported penstock on structural towers built for Alcoa Power Company in North Carolina. The penstock wound through the hills. The roadway you see here was no boulevard but was passable. A circuitous roadway through the rough terrain visible in the background made transportation difficult and costly.

If a roadway is not economically feasible, travelers carried on the erected portions of the pipe or temporary tracks carried on temporary blocking must be used to provide transportation access to the site. Size of tube, weight of pieces and access influence whether erection will be on knocked down or preassembled large section basis. There are cases where the owner eventually builds roadways for maintenance or other purposes after the work is completed. Had these roadways been built in advance of penstock construction, cost of temporary roads could have been eliminated.

When grades are steep and crawler cranes are not maneuverable, overhead cableways or temporary tracks must be used to transport knocked down material or preassemblies to final location.

Figure 1

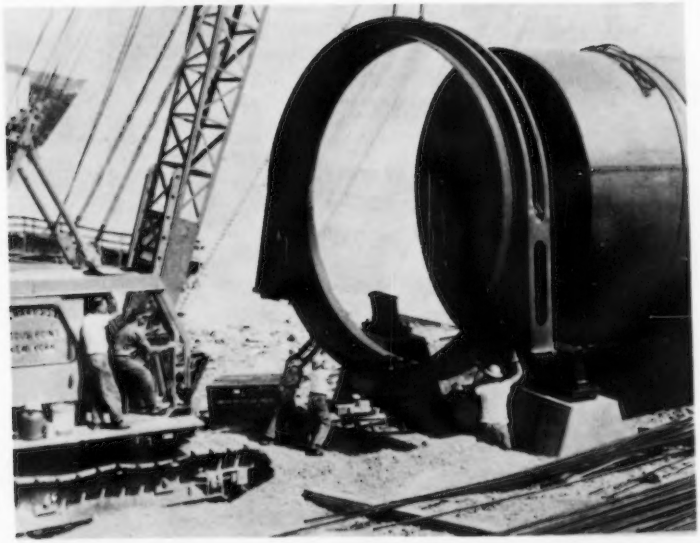


Figure 2

Figure 3

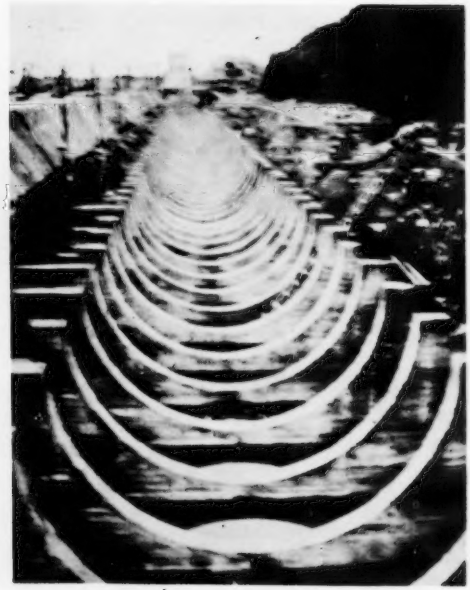


Figure 4

Figure 5

Figure 6

Figure No. 6 - (S-75) shows an 8' diameter ring supported penstock built for Alcoa Power Company in North Carolina. The pipe line in the background was on a 45-degree slope heavily wooded. This section was constructed using track to carry large assemblies to final location. The section in the foreground was supported on structural towers 40' tall. Inaccessibility required erection by brute manhandling of individual rings and cantilever construction.

The cableway is often very suitable in extremely rugged terrain particularly if the alignment is straight. But, the capacity is limited and costs for such gear are high. Usually materials are lowered down grade as it is easier to lower than to lift. This applies to both track or cableway transportation. Roadways are still needed because points of access to the pipe line must be provided to feed materials to cableway or track installations. The piers for cradle supported penstocks could be designed to provide support for trackage thus eliminating considerable cost for track supports.

Penstocks on tall piers (usually long spans on ring supports) present a different problem. The scheme of erection is governed by the ground terrain on which handling equipment must travel. If the terrain is mild crawler cranes can be used to lift pipe sections into final position. Gin poles are sometimes used for raising materials into place but are unwieldy. If the terrain is extremely rough and does not permit ground transportation, cantilever construction with travelers on the pipe must be employed.

Type B

Penstocks in tunnels are another problem and are of two types: i.e., tunnel liners and free standing pipe. Some tunnels are concrete lined while others are in raw rock cuts. The lined tunnel is conducive to good workmanship and makes the steel tube construction job easier. It is usually dry. In tunnel work pipe sections are preassembled and welded into long sections outside of the tunnel and transported into the tunnel on low-boy cars.

Figure No. 7 - shows the preassembly operation near the tunnel portal.

Figure No. 8 - shows the preassembly loaded on the car being moved into the tunnel.

Clearance between the tunnel wall and the pipe is frequently inadequate. This often occurs on tunnel liners that will be fully embedded in concrete. A minimum clearance of 18" between the tunnel wall and the steel liner is needed to properly support and weld the liner. We have had cases where as little as 6" clearance was provided. The small clearance restricted the amount of stiffening on the outside of the liner needed for pressure grouting and external hydrostatic head and complicated the bracing provisions necessary to resist buoyancy. The transportation problem was also difficult due to lack of head room and clearance for pipe carriages. Free standing pipe in tunnels with large ring supports may require as much as 3' of annular clearance to provide adequate access for materials and workmen during construction and access for maintenance of the finished structure.

Type C

Tubes embedded in concrete dams should be preassembled into ring sections in a separate yard to avoid interference with concrete placement.

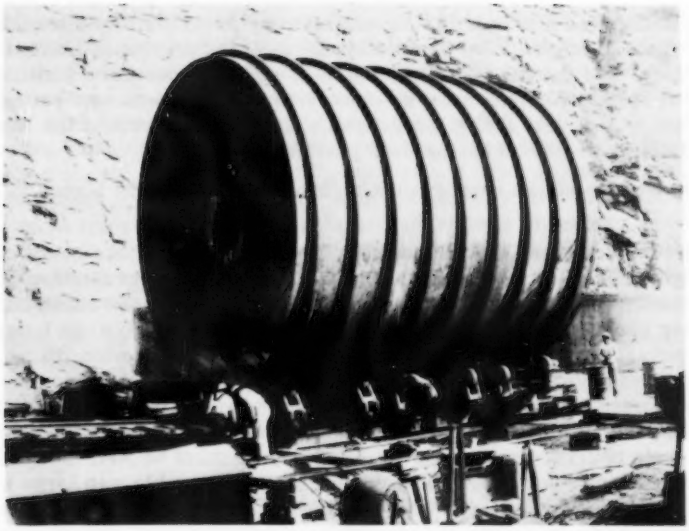


Figure 7

Figure 8

Welding in the dam site is held to a minimum by preassembly into sections several rings in length. The length of preassembled section is limited by the lifting capacity of the concrete handling gear and transportation facilities. Placement of pipe sections must be closely coordinated with concreting operations. A problem facing the penstock builder is support of the tubes so they will not be displaced by concrete placement.

Figure No. 9 - shows supports anchoring a section of pipe placed in the dam.

Tubes on steep inclines require tall supports when concrete is not stepped to closely conform to the slope of the pipe. Work must be performed in the air at considerable loss in efficiency. In addition there is the added cost of scaffolding and support material. It is also more difficult to align tubes on tall supports. It is good practice to preassemble sections to the lifting limit of the concreting gear. This reduces the number of lifts and keeps the down-time of concrete handling equipment to a minimum. This pays off two ways. It reduces costs of handling-equipment rental-time and reduces interference with concrete pouring operations. Welding details must be well thought out so this operation is performed with dispatch. Preassembly into large sections concentrates the majority of welding and radiographing in the pre-assembly yard where it can be performed with greater care and efficiency.

There should be a definite understanding on the amount of internal shoring the owner plans to provide to maintain roundness of sections. If none is

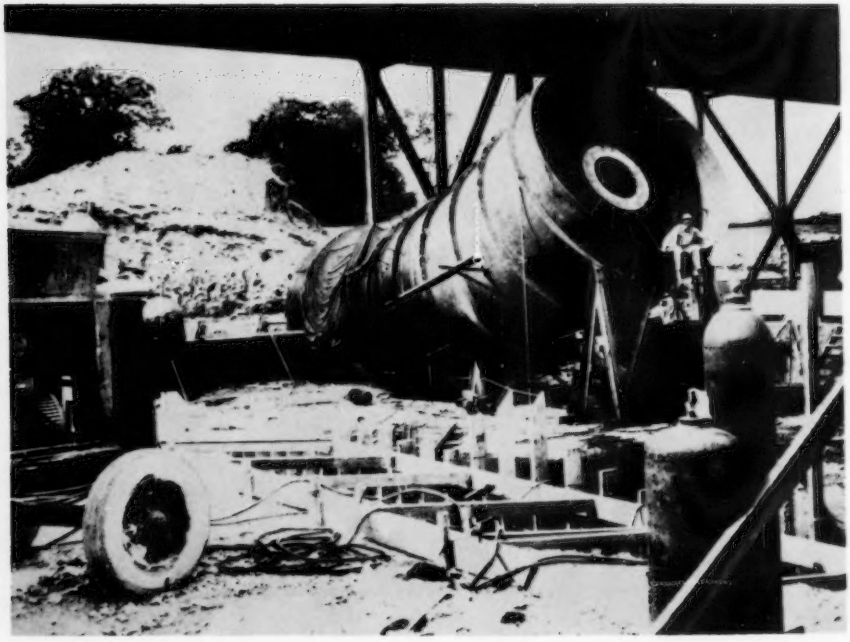


Figure 9

planned and the penstock builder is to provide all the shoring, the specifications should so stipulate. The engineer should outline in detail how concrete will be placed because the amount of shoring required is influenced by the order of concrete placement. Spiders alone are not always sufficient. Some external stiffeners are needed to provide the rigidity needed to withstand concreting pressures. Considerable steel tonnage is involved and someone has to pay for it. It might as well be understood beforehand who will provide it. The degree of rigidity provided is determined by how much out of roundness can be tolerated. In some instances unrealistic roundness requirements are demanded. Diameter difference equal to one percent of theoretical diameter is reasonable. More stringent requirements are beyond normal fabrication and erection limits. Waterway area loss is negligible.

Figure No. 10 - shows calculations of percent area loss by out-of-roundness. The pipe is assumed to be distorted into the form of an ellipse.

Type D

We have had only two contracts for penstocks buried in the ground. One was buried in a soil berm; the other was buried in gravel. The pipe buried in soil had no stiffening and there is no telling what the shape of the tube is now. During formation of the berm a tractor ran over the top and collapsed a section of the pipe. This project probably should have had enough stiffeners to hold shape until the berm was formed. The unit buried in gravel did have adequate stiffening but placement of gravel was performed with a dozer. The pipe was forced laterally out of line by construction of the gravel cover from one side. A gravel cradle should have been hand placed for at least 120° of the invert and the remainder placed with a clam shell from above. As far as is known both projects are operating satisfactorily today.

Temperature movement during construction plagues every penstock builder. Alignment is difficult to maintain when the tube is exposed to direct sunlight. One side is in bright sunlight; the other is shaded. Differential temperature deflects the line generating large side lurch forces on the supports. The problem is magnified on long span ring supported jobs. If erection is performed by the cantilever method the end of the cantilever can fluctuate off course several inches from night ambients to extreme day temperatures. Alignment checks should be made at a time when the temperature differential in the pipe is at a minimum. Checks made during bright sun exposure can be in error.

Figure No. 11 - shows calculations of pipe deflection due to solar effects.

Anchorage at the supports against temperature-side-lurch is a calculable item. Side-lurch and uplift anchorage is needed in the completed penstock as well as during construction. Side lurch forces occur each time the penstock is completely unwatered. Keepers on roller or rocker supports have been stripped off by this force. When a penstock has been unwatered for any reason it is a good practice to inspect the line before refilling to see that no mishap has occurred at any point. High towers carrying a penstock line are also subjected to temperature side-lurch and should be designed to resist this force. The greatest side-lurch force is generated in the second support

PERCENT AREA LOSS DUE TO OUT-OF-ROUNDNESS.

$$A_c = \pi R^2$$

$$A_e = \pi ab = \pi(R^2 - \Delta R^2)$$

$$\Delta A = A_c - A_e$$

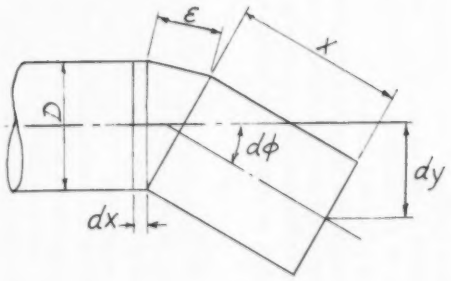
$$\% \text{ Area Loss} = \frac{\Delta A}{A} \times 100 = \frac{\Delta R^2}{R^2} \times 100$$

$$k = \frac{\Delta R}{R}, \quad \% \text{ Area Loss} = k^2 \times 100 = \frac{K^2}{4} \times 100$$

Where: A_c = Area of circle.
A_e = Area of ellipse.
ΔR = Radius change from theoretical.
$k = \frac{K}{2}$
K = Ratio of diameter difference to theoretical diameter.

Figure 10

PIPE DEFLECTION DUE TO SOLAR EFFECTS

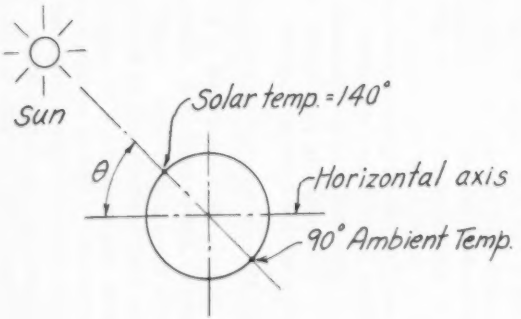


$$d\phi = \frac{\varepsilon}{D} = \frac{\Delta T\, C\, dx}{D}$$

Where: ΔT = Temp. difference = 50°F average

C = Coeff. of expansion = .0000065 in/in/°F

dx = Differential length.



Cross Section of Pipe

$$y = \int_0^L \frac{\Delta T C x\, dx}{D} = \frac{\Delta T C}{D} \cdot \frac{L^2}{2}$$

Depending on position of sun:

$$y_V = \frac{\Delta T C L^2}{2D} \sin\theta = \text{Vert. movement}$$

$$y_H = \frac{\Delta T C L^2}{2D} \cos\theta = \text{Hor. displacement}$$

Figure 11

removed from an expansion joint. On large diameter penstocks this force can be several hundred kips in magnitude.

Figure No. 12 - shows calculations of fixed end moments in each span due to solar effects.

The reactions at each support can be determined by moment distribution or other methods.

During construction of the penstock, frequent station checks must be made for growth or shrinkage of the assembly. The length is corrected to mean temperature for the locality. Checks are best made at dawn or after sunset when the steel temperature is uniform over the whole circumference. The length check is important for proper setting of the expansion joint and location of ring girder supports. If the growth or shrinkage exceeds tolerable limits, the length of penstock must be corrected to fit concrete pier locations built in advance of the pipe line installation. Adjustment is made by insertion of a dutchman when pipe length is short and cutting away excess pipe when length is too great.

Shrinkage allowance is an experience item and varies with welding procedures and details used. Shrinkage may be as large as 3/32" for each butt welded joint. Proper allowance in the fabricated material can compensate in part for shrinkage. On long continuous runs the finished length is not exactly predictable.

This poses a problem for the erector when concrete piers are poured to precise station locations in advance of steel erection. On ring supported lines ring stations must match pier positions. There must be some adjustment in the base plates to allow for reasonable misalignment. It is advisable to provide for 2" of misalignment in all directions at each pier. Enlarged holes provided in the base plates will permit shifting the plates to match the rings on the pipe. To secure the plates against shifting, close fitting washers can be dropped over the anchor bolts and welded to the base plates. Provision for such adjustment reduces the number of shrinkage or growth corrections that must be made by the builder on long penstock runs.

Long exposed penstock runs between fixed anchors require expansion joints to relieve temperature stresses. There is no set rule for frequency or spacing of expansion joints. Six hundred feet between fixed anchors is considered to be reasonable spacing for which it is possible to design and build relatively simple expansion units.

Figure No. 13 - shows such an expansion joint.

For such lengths total movement of 6" have been measured. Joints should be located approximately at mid-distance between fixed anchors so each leg moves the same amount - about 3" maximum. This 3" dimension affects rocker design which will be discussed later. Packing gland type of expansion joints are needed for movements as great as 6". Before the advent of stainless steel, sliding surfaces on the joints were just plain carbon steel. The plate surfaces were burnished sufficiently to remove rough edges, projections and loose mill scale. The surfaces were not smooth and were subject to corrosion despite the generous use of lubricant in the packing.

When stainless steel became available the first attempt to provide corrosion free sliding surfaces was by means of plug welding 16-gage sheets of

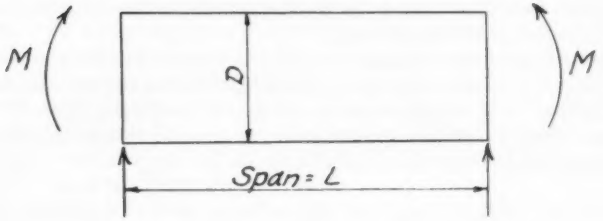
CALCULATION OF FIXED END MOMENTS DUE TO SOLAR EFFECTS.

$$d\phi = \frac{\Delta T C dx}{D}$$

$$\frac{M dx}{EI} = d\phi$$

If I is constant throughout the length of a span:

$$M = \frac{EI}{D} \Delta T C$$



Calculate reactions by moment distribution or other methods.

Figure 12

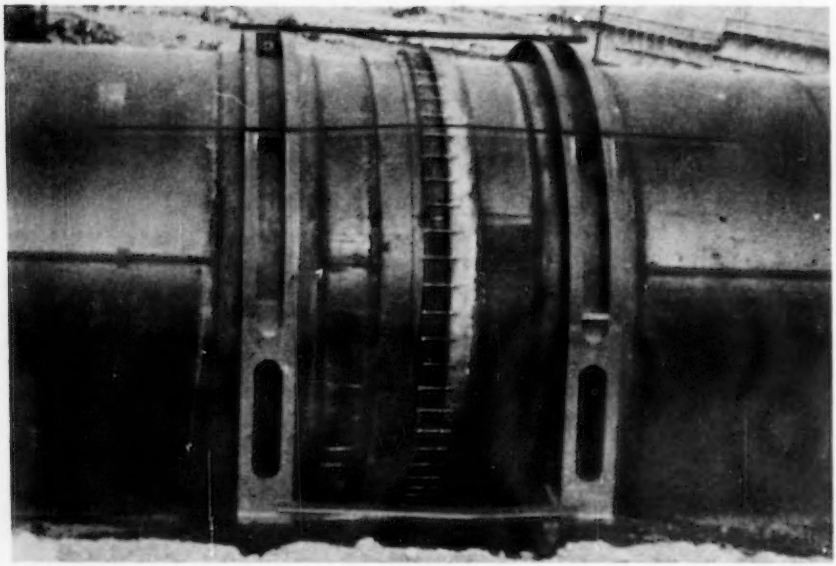


Figure 13

stainless steel tightly wrapped around the carbon steel plates. This was an improvement but the plates billowed and buckled because of dissimilar coefficients of expansion. Later when clad plates were developed and bonded stainless steel surface required only grinding and polishing the welded joints. This practice is still in vogue today.

If economical expansion joints are expected, lathe turned workmanship should not be demanded, in fact it cannot be justified. Good serviceable expansion joints with little or no leakage can be produced by normal plate fabrication methods. If lathe turned workmanship is demanded the cost will approach machinery prices.

Rockers and Rollers

Limiting maximum movement of each leg of the pipe to 3" because of temperature variation results in economical roller design. The design details are compact and easily shielded. We prefer to use built-up weldments for rollers rather than castings because we can control the quality of the finished units. Weldable metals are now available having properties that formerly were only available in castings. Rockers require precision finished journals and pins and are subject to seizing even when designed to low bearing unit pressures, unless dissimilar metals are used. Precision machining is costly which most penstock work does not warrant; furthermore, precise installation is not possible especially on large pipe. Rollers on the other hand do not require precise machining or precise installation; they are doing as good a job as rockers. Rollers or rockers must be accurately positioned when they are set to avoid fouling. Tilt must be carefully calculated and the roller positioned accordingly.

Figure 14 - shows calculated roller settings which are given to our construction crews to properly set the rollers for existing ambient temperatures. This is an important phase of construction and must be performed with great care.

Hydrostatic tests are usually specified. The purpose of the test should be stipulated in the specifications. A tightness test is interpreted to mean tightness under service conditions. It is best performed when the penstock unit is completed. No special heads are needed for its performance. The pipe is simply filled with water and inspected for leaks.

An overload strength test is another matter. If the test is made on the completed pipe unit, special test heads must be installed in the intake, the surge tank riser, and in the pipe just above the scroll case. The use of control valves as a substitute for the lower test head has not always been satisfactory. Leakage through valves is so great, expensive test pumps must be provided to maintain specified test pressure.

Figure No. 15 - shows a test setup used to perform an overload strength test. This diagram plus complete instructions are given to the construction superintendent to guide him in the performance of the test.

During overload test expansion joints must be tied to avoid overloading anchor blocks by the additional thrusts imposed on them. Tying the expansion joints introduces unwanted temperature stress loads on the anchor blocks especially when test water temperature differs from ambient temperatures, or when the ambient temperature varies while the joint is tied. This effect should be carefully studied before an overload test is specified on a completed penstock installation.

If an overload water test is wanted to test longitudinal seams only, it can be performed in the assembly yard. On small pipe the test gear can be quite simple. On large pipe the gear becomes tremendous and expensive. Portable universal test gear to fit all pipe diameters would be prohibitive in cost. Test gear is usually devised for each job and scrapped on completion of the project. Only mechanical items, such as pumps, are salvaged for future use. In some instances it is more economical to weld test heads on preassembled sections and cut them off for reuse on completion of the test. This is feasible only on small projects.

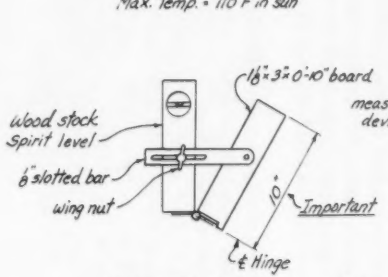
Special fittings such as wyes and tees are usually tested individually. It should be recognized service conditions cannot always be duplicated in the overpressure test. Longitudinal loads are developed under test conditions that may not be present under working conditions. As a result the overpressure test of fittings is not always a true measure of their strength under operating conditions. In every case where overpressure tests are specified the owner must be prepared to accept added costs for this operation.

Radiographing is a part of welding. Welding is a subject in itself. Its scope is too vast to be properly covered in this paper. A few pertinent remarks will be made concerning this operation.

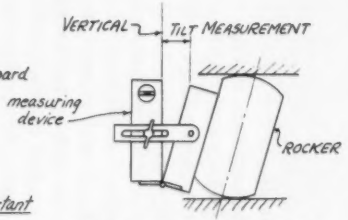
When extremely heavy plates are required in special details, such as reinforcing members in wyes and tees, consideration should be given to quality materials. Except for a few refinements, the steel materials used in penstocks have not changed materially in the past forty years. For reasons of economy structural quality steel plates were generally used.

18'ϕ × 667' LONG PENSTOCK FOR NIAGARA MOHAWK POWER CORP. SOUTH COLTON, NEW YORK. CONTRACT 7-0806.

Mean Temp = 45°F
Min. Temp. = -20°F
Max. Temp. = 110°F in sun



SPECIAL MEASURING DEVICE



METHOD OF SETTING ROCKERS

EXPANSION JOINT MOVEMENTS FOR VARIOUS AIR TEMPERATURES.

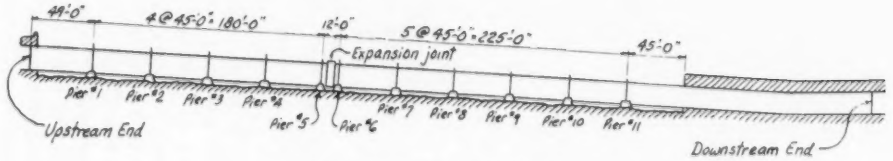
AIR TEMPERATURE °F						
35°	45°	55°	65°	75°	85°	95°
+13/32"	0"	-13/32"	-13/16"	-1 3/16"	-1 19/32"	-2"

Note: Clearance = 7" @ 45°F (See Dwg. #3)
(+) dimension is increase in clearance.
(—) dimension is decrease in clearance.

TILT MEASUREMENTS FOR VARIOUS AIR TEMPERATURES.

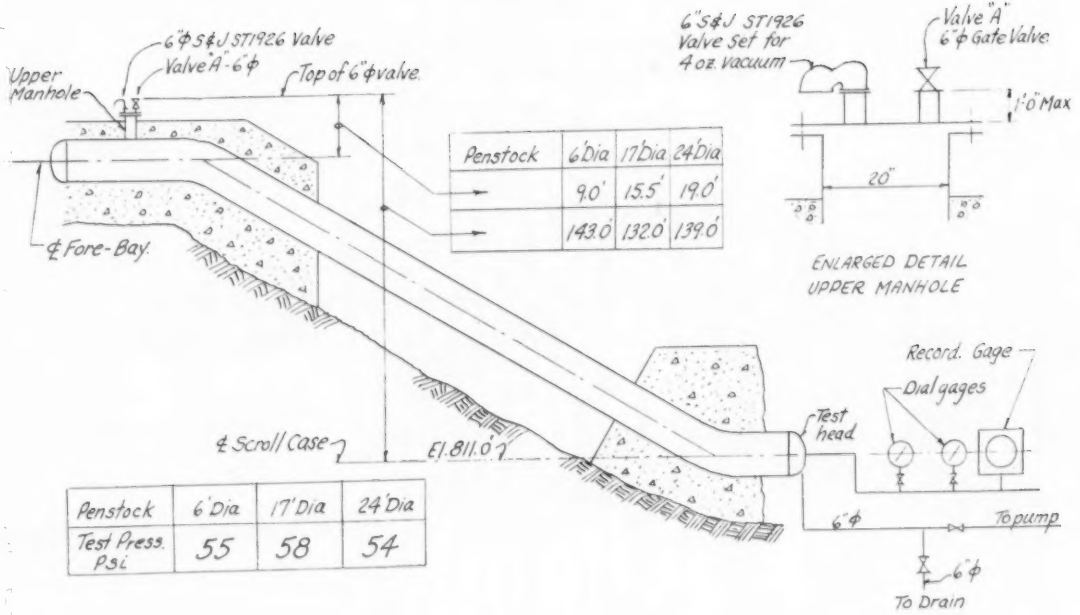
PIER NO.	AIR TEMPERATURE °F						
	35°	45°	55°	65°	75°	85°	95°
1	19/32"	5/8"	21/32"	11/16"	23/32"	3/4"	25/32"
2	9/16"	5/8"	11/16"	3/4"	13/16"	27/32"	29/32"
3	17/32"	5/8"	23/32"	25/32"	7/8"	31/32"	1 1/16"
4	1/2"	5/8"	23/32"	27/32"	31/32"	1 3/32"	1 7/32"
5	15/32"	5/8"	3/4"	29/32"	1 1/16"	1 7/32" ↑	1 11/32"
6	25/32"	5/8"	7/16"	1/4"	3/32"	3/32" ↑	1/4" ↑
7	3/4"	5/8"	15/32"	5/16"	5/32"	0"	1/8" ↑
8	23/32"	5/8"	1/2"	3/8"	1/4"	1/8"	0"
9	11/16"	5/8"	17/32"	7/16"	11/32"	1/4"	5/32"
10	21/32"	5/8"	9/16"	1/2"	7/16"	3/8"	5/16"
11	21/32"	5/8"	19/32"	9/16"	17/32"	1/2"	15/32"

Note: Tilt measurements marked ↑ are upstream from vertical.
All other tilt measurements are downstream from vertical.



ELEVATION VIEW OF PENSTOCK.

Figure 14

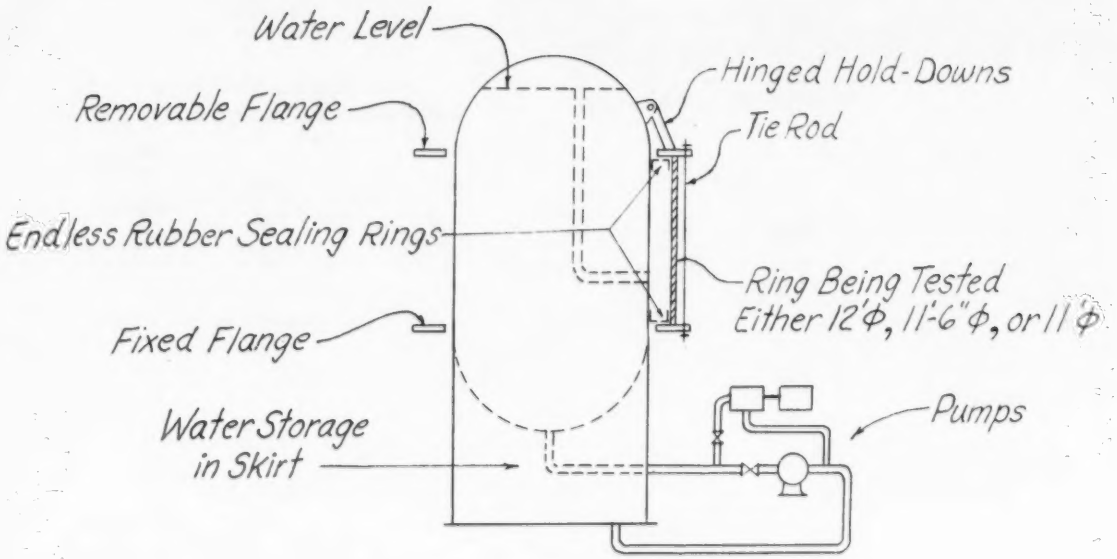


Penstock	6' Dia	17' Dia	24' Dia
Test Press. P.S.I.	55	58	54

Test pressure is pressure above initial gage reading psi.

TEST HOOK-UP FOR PENSTOCKS
CONTRACT 6-2800
CENTRAL MAINE POWER CO.
INDIAN POND, MAINE

Figure 15



HYDROSTATIC TEST GEAR

Figure 15A

These materials served very well. The growing demand for power necessitated development of higher heads and increased flows. Steel plate thicknesses increased correspondingly. Formerly plate thicknesses greater than 3/4" were seldom required. Today it is not unusual to meet with requests for penstock plates 2" in thickness. Because some welding difficulties were encountered with thick plates, radiographic examination was deemed advisable. Some designers became panicky and extended this examination to practically all plate thicknesses even as low as 5/16". There is little justification for radiographic examination of welds in plates less than 5/8" in thickness. Improved electrodes and welding techniques and the experience with a countless number of structures built without full radiographic examination support this statement.

To control welding quality we formerly used trepanning where 100% radiographic examination was not specified. Today we use the spot x-ray to control welding quality. It is a small portable unit available from manufacturers of radiographic equipment.

Figure No. 16 - shows a spot radiograph. Its plate thickness capacity is 1-1/2 inches maximum under 8 minute exposure.

Figure No. 17 - shows the portable unit in use on a tank shell.

Figure No. 18 - shows component parts of the unit.

Complicated details such as are encountered in wyes and tees justify consideration of premium materials, radiographic examination where feasible, and in some cases even thermal stress relief. Because of his experience, the builder is sometimes compelled to provide these special elements at his own expense to assure safe construction. Weld metal should be used to join metals together, the purpose for which it is intended. It should not be used as a plaster to fair out irregularities to achieve streamlined flow. Such misuse invites weld cracking troubles that are frequently irrepairable.

Wherever possible simplicity of design should be maintained to assure ease of execution by the builder and performance of the finished structure in accordance with design assumptions.

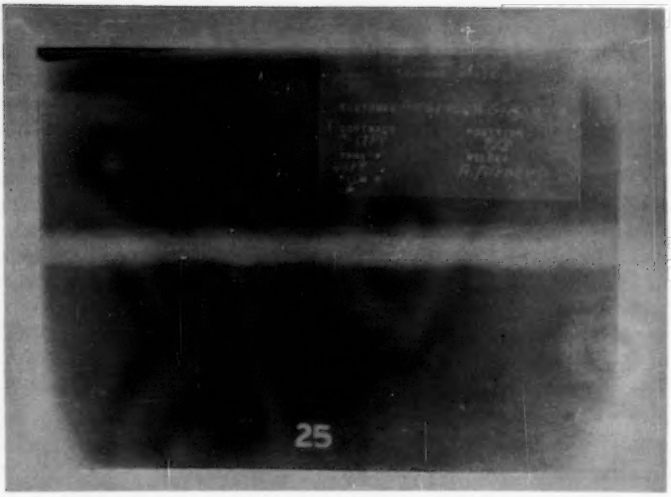


Figure 16

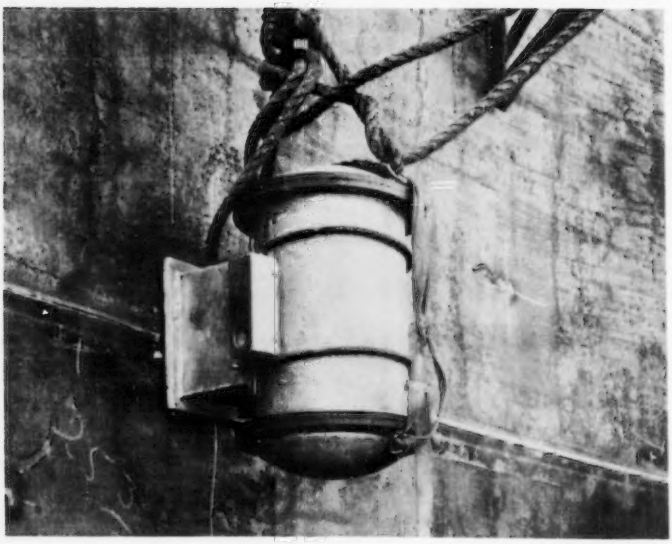


Figure 17

Figure 18.

Paper 1285 PO 3

Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

PENSTOCK DESIGN AND CONSTRUCTION[a]

G. R. Latham,[1] M. ASCE
(Proc. Paper 1285)

ABSTRACT

General design features and construction procedures for various penstock installations are described and illustrated in this paper.

The design and construction of penstocks in accordance with the practice outlined result in adequate installations at reasonable cost.

The purpose of this paper is to describe general design features and construction procedures of several installations which are representative of our company's practice. Many pipe lines have been designed and constructed which have given satisfactory service and it is believed that this practice, based on 50 years experience, results in adequate and practical design at reasonable cost.

Current penstocks are of welded construction and in general the plate is specified as ASTM Specification 285 Grade C but plate meeting Specification A7 and A-283 have been accepted on occasions to expedite delivery. Design is normally based on an allowable unit stress of 18,000 lbs/sq in. with a weld joint efficiency of 85 per cent. A so-called "corrosion allowance" as adding an arbitrary amount to the plate thickness is not used but a 3/8 in. minimum plate thickness is provided. Best corrosion resistance is obtained by thorough cleaning of the plate and applying a good protective coating. For our latest penstocks commercial sand blasting and one shop and one field coat of zinc-rich paint is usually recommended.

American Welding Society Specifications are followed and all welders are required to be qualified accordingly. When automatic welding is used, representative test coupons are taken and subjected to conventional bend and tension tests. Manually welded joints are usually checked by means of spot

Note: Discussion open until November 1, 1957. Paper 1285 is part of the copyrighted Journal of the Power Division of the American Society of Civil Engineers, Vol. 83, No. PO 3, June, 1957.

a. Paper to be given at the "Symposium on Penstocks" to be presented at the Buffalo, New York, meeting of the ASCE, June 3-7, 1957.

1. Chf. Structural Engr., Ebasco Services Inc., New York, N. Y.

X-ray in preference to trepanning. Full X-ray examination of all welds is not specified. Shop assembled pipe sections having plate over 1 in. thick and shop assembled wyes or other special sections are usually stress relieved and hydraulically tested to 150 per cent of the total head in the shop. Unless an expansion joint is provided the final closure welds between anchors are specified to be made at temperatures of about 50° to minimize locked up tensile stresses. Due to their maintenance requirements expansion joints are kept to a minimum. Ample size manholes are provided for access. Completed installations are generally tested by filling with water under static head and examining for any leakage. It is not customary to provide an over pressure hydrostatic test of a completed installation.

The following installations are examples of different types of construction, designed to suit specific conditions. The accompanying photographic slides illustrate the construction.

Upper Malad Hydro-Electric Development - Idaho Power Company - 1946

This is an example of simple construction consisting of an exposed steel pipe with stiffeners, supported on concrete saddles. An inclined 10 ft diameter penstock about 225 ft long supplies water from an intake structure to an 8,000 kw unit. (See Fig. 1A.)

The total design head is 180 ft including water hammer. The penstock shell is 3/8 in. plate with 6 x 4 x 3/8 in. angle stiffeners at about 7 ft centers and is supported on 120° concrete saddles at about 21 ft centers. (See Fig. 1B.) A conventional packed sleeve expansion joint is provided at the upper end of the penstock. The pipe was shop fabricated in sections about 21 ft long which resulted in simple erection by the field construction force. Good competition among fabricators is usually obtained on this type of construction and its complete installed cost has proved economical.

Kerr Hydro-Electric Development - Montana Power Company - 1954

This installation is an example of a penstock liner in a rock tunnel. The powerhouse is located against a solid rock cliff and the water is supplied through a non-reinforced concrete lined tunnel. The welded steel liner extends about 100 ft into the tunnel and connects to the riveted spiral casing.

The design head is 278 ft including water hammer. The liner is 20 ft diameter and is constructed of 15/16 in. thick plate. It was shipped in curved segments and erected into rings in an assembly yard adjoining the site. Internal spider stiffeners were provided for handling and concreting, eliminating external ring stiffeners which would also hinder the concreting. Two rails on concrete piers were provided in the tunnel for installing the liner ring which had two rail shoes on each side of the pipe. The first 8 ft long pipe ring was lowered into place on the rails at the face of the cliff and then pulled part of the way into the tunnel, resting on the rails. (See Fig. 2A.) The next ring was lowered into place and welded to the first ring after which the two rings were pulled part way into the tunnel and next ring attached. This process was repeated until all the rings were welded together and

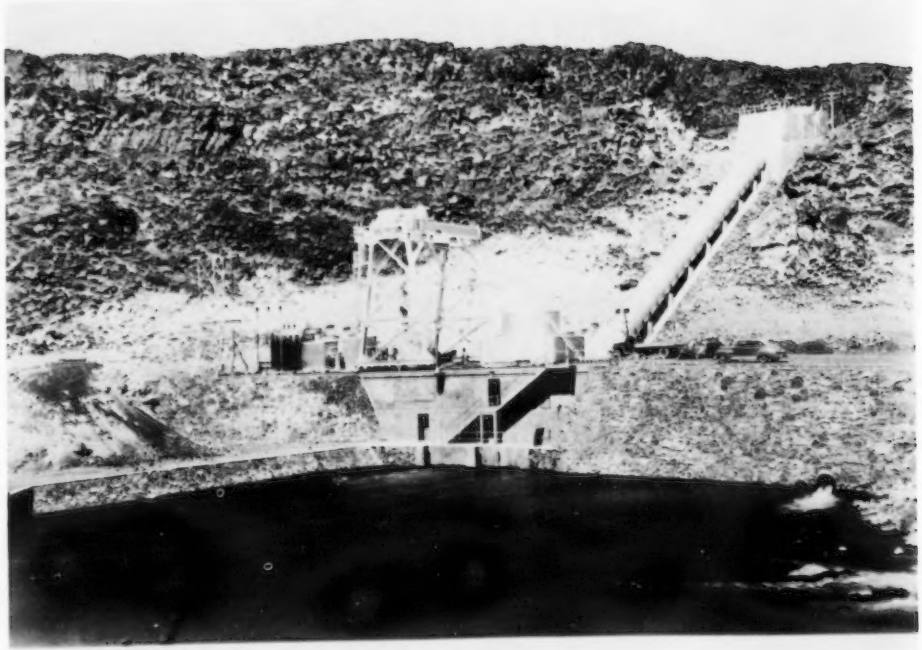


FIG. 1 A

UPPER MALAD HED

GENERAL VIEW

FIG. 1 B

UPPER MALAD HED

PENSTOCK

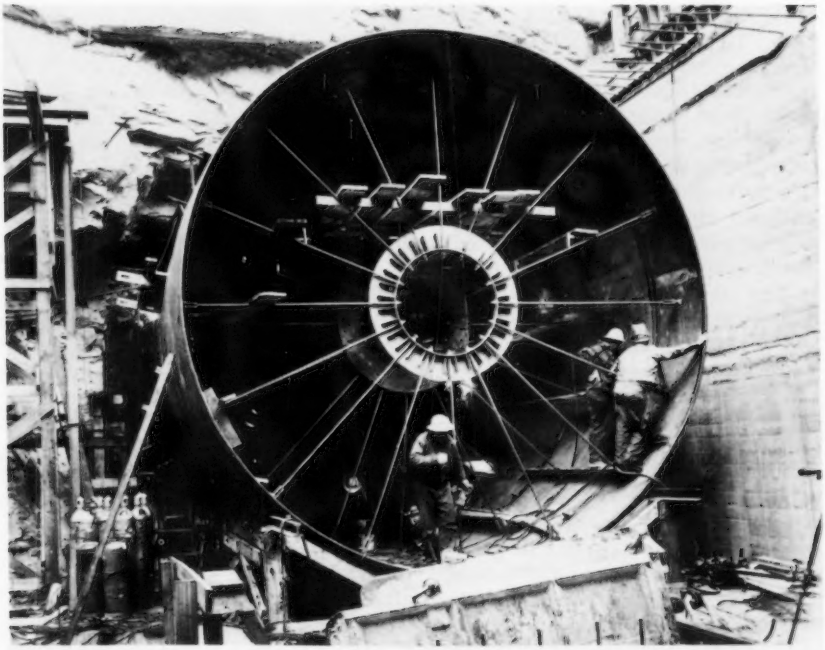


FIG. 2 A

KERR HED

SECTIONS AT TUNNEL ENTRANCE

placed in final position. (See Fig. 2B.) Water stop rings are provided at the upper end of the lining and cable tiedowns at 6 ft intervals prevented floating of the liner during concreting. Concrete was placed by the pumpcrete method and later the tunnel crown was pressure grouted. The connection of the penstock to the riveted spiral casing was also done by riveting.

The liner fabrication and erection was done by the American Pipe and Construction Company of Portland, Oregon.

C.J. Strike Hydro-Electric Development - Idaho Power Company - 1952

This installation is an example of a buried penstock construction. The dam was of earth fill on a soil foundation and three 22 ft diameter inclined penstocks approximately 350 ft long were provided between the intake and the powerhouse. (See Fig. 3A.) Investigation of various types of construction indicated a very substantial economy by using a buried penstock with stiffener rings, uniformly supported on the soil. This eliminated large size ring girders with extensive concrete supports or saddles which would have been required with other methods.

The pipe was designed for various loading conditions including pipe empty, filling and backfill cover. The design head is 120 ft including water hammer and the pipe shell varies from 3/8 in. to 5/8 in. with 13 in. deep stiffeners on 4 ft centers. (See Fig. 3B.) The material was shipped in curved segments and assembled into rings at the site. The stiffened pipe rings were unusually substantial and did not require any spiders during handling and backfilling. (See Fig. 3C.) A special diaphragm type expansion joint was installed at the midpoint and its flexibility also allows for possible differential settlement between the intake structure and the powerhouse. The pipe exterior was sand blasted and a hot waterworks enamel coating was applied. Only one coat of red lead paint was applied to the penstock interior as the owner's long experience with previous installation on the Snake River had shown that the water was not corrosive. Actually a protective whitish deposit usually accumulates on the pipe interior.

It was not possible to inspect the exterior of the completed penstock for leaks because it was necessary to backfill the pipe to provide proper support for filling. Therefore, in addition to trepan plugs for weld quality control, all welds were inspected by magna-flux to insure against any possible flaws.

The general fill was graded gravel with a minimum of 18 in. of sand placed against the penstock. The sand fill was sluiced into place and care was taken to load both sides of the penstock equally to avoid displacement and unnecessary stresses. (See Fig. 3D.) The major portion of the pipe was backfilled and the temperature stabilized before the final closure welds were made.

Indicator rods which projected above the fill were placed at intervals at top of the penstocks. Readings were taken when the penstocks were filled and only very minor deflections resulted. There is a pipe and catch basin draining the backfill and the flows were checked as each penstock was filled. No change in the drainage flows were observed indicating that the penstocks were water tight.

The pipe was fabricated and erected by the Chicago Bridge & Iron Company.

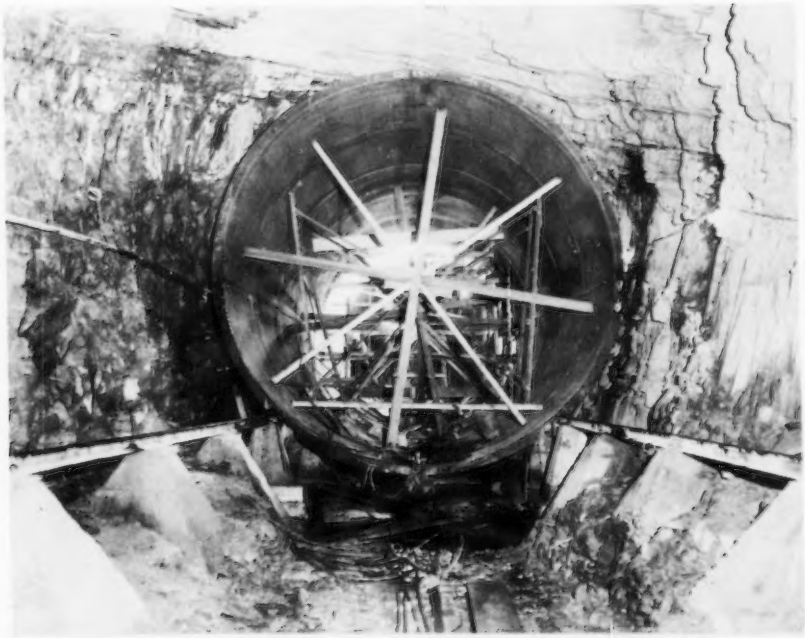


FIG. 2 B

KERR HED

SECTIONS IN TUNNEL

FIG. 3 A

C.J. STRIKE HED

GENERAL VIEW

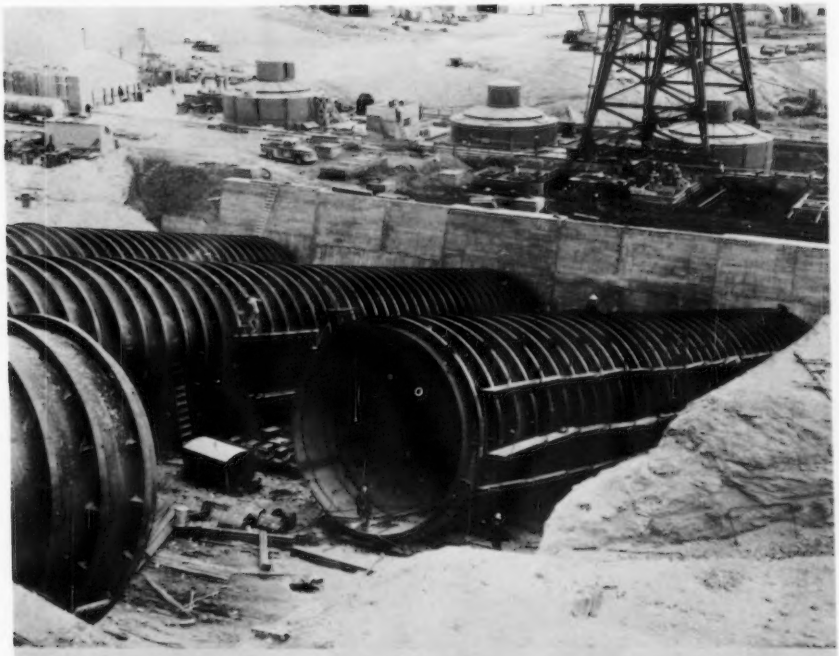


FIG. 3 B

C.J. STRIKE HED

PENSTOCK ERECTION

FIG. 3 C

C.J. STRIKE HED

PENSTOCK ERECTION

FIG. 3 D

C.J. STRIKE HED

PLACING BACKFILL

Paucartambo Hydro-Electric Development - Cerro De Pasco Corp - 1955

This installation is an example of a high head penstock using Dresser coupled field joints. The project is located on the Paucartambo River in the Department of Junin in Peru, South America and was built for the Cerro de Pasco Corporation to provide power for their mining and smelting operations. The design was developed in cooperation with the Engineering Department of the Cerro de Pasco Corporation. To provide access it was necessary to improve about 43 kilometers of existing road and construct about 60 kilometers of new road. This is a two lane road with a spectacular gradient, located high in the Andes Mountains, and cost about five million dollars.

The penstock arrangement selected after extensive studies was a single pipe supported on concrete saddles, with two laterals at the lower end to serve the three units of 24,000 kva capacity each. A tunnel approximately 8 miles long, mostly concrete lined, carries the water from the dam and terminates in a surge chamber excavated in the rock. The upper end of the penstock is connected to the tunnel by a plate steel liner having a rock trap composed of a steel screen with 1-1/2 in. openings and a collecting hopper with a sluice valve to discharge debris deposited by the tunnel flow. A 6 ft diameter butterfly valve, provided as an emergency shut off at the upper end of the penstock, is equipped with an automatic protective device to close with excessive velocity. Air valves are also provided to prevent vacuum under unusual flow conditions. The steel penstock is approximately 4500 ft in length and drops about 1550 ft in elevation with a maximum design head of 1850 ft and a gradient of about 40°. (See Fig. 4A.) The penstock varies from 80-5/8 in. OD with a 7/16 in. shell plate at the tunnel connection to 74-5/8 in. OD with a 1-13/16 in. thick shell plate at the lower laterals.

The penstock was shop welded in sections about 30 ft long to conform to shipping limitations. Dresser coupled field joints were selected as the most economical and had the following advantages: -

1. Eliminated the difficulties of obtaining qualified welders in this remote location.
2. Eliminated field stress relieving of heavy field welded plates.
3. Eliminated expansion joints.
4. Dresser couplings allow a limited deflection between adjacent pipe sections which permitted the penstock to follow the ground contour and reduced the number of vertical curves which required thrust blocks.
5. Better ability to adjust to minor displacements from possible earthquake shocks.

The penstock was furnished by the Bethlehem Steel Export Corp and fabricated at Steelton, Pennsylvania. It was shipped by boat to Lima, Peru and then by railroad and truck to the job site. The laterals and straight pipe sections constructed of plate 1 in. thick and over were stress relieved by annealing and then hydrostatically tested to 150 per cent of design pressure in the shop. The sections were nested to save shipping charges and to prevent damage during shipment a protective angle ring and steel spider were attached to the pipe ends which had been fabricated to close tolerances for

FIG. 4A

PAUCARTAMBO HED

GENERAL VIEW

the Dresser couplings. These precautions proved effective as the only serious damage experienced was due to the dropping of one pipe section during unloading from the ship.

A 25 ton capacity inclined cable runway for transporting equipment to the powerhouse, parallels the penstock alignment and it was used for installing the penstock sections. One hundred and twenty degree concrete saddles are located on each side of the Dresser couplings to support the pipe. One saddle was an anchor and the other was a sliding type and both were constructed on a common base. The concrete saddles were constructed in advance and were placed by Pumpcrete Machine; concrete anchor blocks at bends were poured after the pipe was erected. Excess penstock length was provided for "makeup" between anchor blocks.

The construction force developed a very satisfactory erection procedure which started at the powerhouse and proceeded uphill. The Dresser coupling rings were put on the pipe section and it was then loaded on the cable car. The car was then lowered to approximate position and tied to the rails. To prevent the pipe section from sliding down the steep slope when being moved sideways to the saddles an auxiliary hoist cable was fastened to it. The pipe sections were rolled from the car by means of jacks and rope "hold backs." (See Figs. 4B to 4E, incl.) No elaborate special equipment was required and an efficient crew was developed from local labor. The normal crew comprised about 30 men and they averaged installing one penstock section in an 8 hour day. The time required for unloading the pipe from the car to the saddles varied from 40 man-hours on the steep inclines to 15 man-hours on a slight incline. The erection of the penstock proceeded very satisfactorily and no undue difficulties were experienced. After completion the construction force reported that the Dresser coupling method was simpler and faster than field welding and had proved to be a very economical installation.

Trombay Thermal Power Station - The Tata Power Company - 1956

This is an unusual installation of a pipe line supplying circulating water to a steam electric station at Bombay, India and has been included as it is believed of interest. It was necessary to use sea water from the bay of Bombay which required constructing a concrete pile dock about 800 ft long to cross the shallow tidal area to the intake. This was a caisson structure in deep water on which the circulating pumps were located.

The pipe is 6 ft diameter and is about 4200 ft long with a 7/16 in. shell plate and 4 x 3 x 3/8 in. angle stiffeners on about 6 ft centers. The pipe was designed for an internal working pressure of 60 lbs/sq in. and a possible vacuum of 15 lbs/sq in. As protection against the severe corrosive effect of the sea water the pipe interior had a 1 in. cement lining. This was applied by spinning process after fabrication and before shipment to the site. Zinc anodes protected the exposed steel at the expansion joints against corrosion.

The portion of the pipe on the dock was erected with field welded joints and was supported on 120° concrete saddles at 20 ft centers to suit the dock bents which had already been constructed. The dock was jointly owned with an oil company and supported their pipe lines also. Anchor blocks and conventional packed sleeve type joints were provided to allow for expansion.

FIG. 4 B

PAUCARTAMBO HED

UNLOADING PENSTOCK SECTIONS

FIG. 4 C

PAUCARTAMBO HED

PLACING PENSTOCK SECTIONS

FIG. 4 D

PAUCARTAMBO HED

PLACING PENSTOCK SECTIONS

FIG. 4 E

PAUCARTAMBO HED

COMPLETED LINE

The major portion of the pipe line was built on a dike or "bund" which had been constructed of rock fill. Dresser coupled field joints were used to allow for possible unequal settlement of the footings. The pipe was fabricated in 42 ft long shop sections and supported by a concrete saddle on each side of the Dresser coupling which was built on a common base. One saddle was an anchor and the other permitted sliding by means of a thin graphited asbestos sheet. (See Fig. 5A.)

The pipe also spanned a canal with a 400 ft long skewed crossing having piers at 60 ft spacing. This portion of the pipe was built with field welded joints and was supported at the piers by ring girders having large radius rocker supports and a sleeve type expansion joint to allow for expansion. This construction eliminated any supporting bridge structure. (See Fig. 5B.)

The pipe was fabricated by the Structural Engineering Works, Limited of India, with steel plate produced in that country.

Pelton Hydro-Electric Development -
Portland General Electric Co — 1957

This installation consists of three buried penstocks connecting the arch dam and the powerhouse and is now being constructed. The dam and powerhouse are on bed rock and the area between is backfilled with compacted sand and gravel fill. The penstocks are supported on this backfill and are buried under a 16 ft depth of cover. The switchyard with the main transformers is located on this filled area over the penstocks. (See Fig. 6A.)

The design head is 180 ft including water hammer and the pipe shell is 1/2 in. plate with 10 in. deep stiffeners on 4 ft centers. The penstocks are short, being only about 110 ft long with approximately 50 ft being embedded in the backfill. There is no expansion joint and the penstock is designed for the temperature stresses. It is intended that the final closure weld will be made from the inside of the pipe after the backfill has been placed and the temperature stabilized. The penstock exterior surface is sand blasted and coated with hot waterworks enamel. The selected fill is carefully placed and compacted in uniform layers to eliminate settlement.

A similar design is also being used for two penstocks at the North Fork Hydro-Electric Development under construction for the Portland General Electric Company. Penstocks for both projects are being furnished by the American Pipe & Construction Company. The pipe was constructed in cylinders at the fabricator's plant and hauled about 120 miles to the job site in a special truck rig. (See Fig. 6B.)

FIG. 5A

TROMBAY STATION

C.W. PIPE

FIG. 5 B

TROMBAY STATION

C.W. PIPE

FIG. 6A
PELTON HED
CROSS SECTION

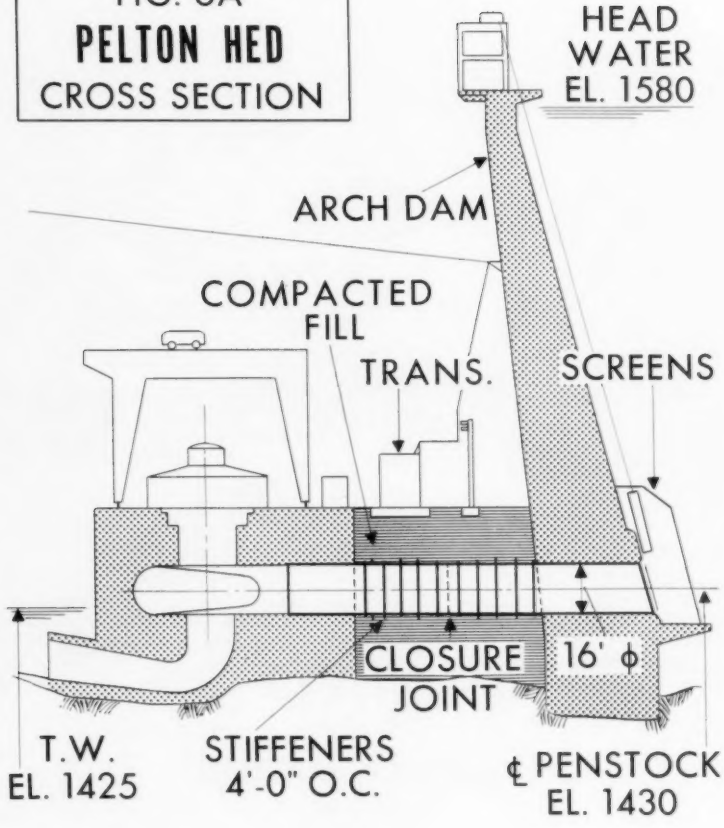


FIG. 6 B

PELTON HED

SPECIAL HAULING RIG

Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

ARCH DAMS: ECONOMY OF CONCRETE DAMS

Louis G. Puls,[1] A.M. ASCE
(Proc. Paper 1286)

FOREWORD

This paper is one of a group presented at the ASCE Symposium on Arch Dams, June, 1956, at Knoxville, Tennessee.

Since the last symposium on masonry dams (April, 1939), much progress has been made in the design and construction of arch dams and their appurtenances. This Symposium was planned to enable engineers concerned with arch dams to exchange their ideas and experiences for the benefit of all.

At this time, it is now known exactly how many papers will be printed from the Symposium. So far, eighteen papers have been approved: "Arch Dams: Their Philosophy," by Andre Coyne (Proc. Paper 959); "Arch Dams: Trial Load Studies for Hungry Horse Dam," by R. E. Glover and Merlin D. Copen (Proc. Paper 960); "Arch Dams: Portuguese Experience with Overflow Arch Dams," by A. C. Xerez (Proc. Paper 990); "Arch Dams: Theory, Methods, and Details of Joint Grouting," by A. Warren Simonds (Proc. Paper 991); "Arch Dams: Santa Giustina Single-Curvature Arch Dam," by Claudio Marcello (Proc. Paper 992); "Arch Dams: Measurements and Studies on Santa Giustina Dam," by Claudio Marcello (Proc. Paper 993); "Arch Dams: The Reno Di Lei Double-Curvature Arch Dam," by Claudio Marcello (Proc. Paper 994); "Arch Dams: Isolato Double-Curvature Arch Dam," by Claudio Marcello (Proc. Paper 995); "Arch Dams: Rio Freddo Dam with Gravity Abutments and Cut-Offs," by Claudio Marcello (Proc. Paper 996); "Arch Dams: Design and Observation of Arch Dams in Portugal," by M. Rocha, J. Laginha Serafim, and A. F. da Silveira (Proc. Paper 997); "Arch Dams: Development in Italy," by Carlo Semenza (Proc. Paper 1017); "Arch Dams: Design of the Kamishiiba Arch Dam," by C. C. Bonin and H. W. Stuber (Proc. Paper 1018); "Arch Dams: Observed Behavior of Several Italian Arch Dams," by Dino Tonini (Proc. Paper 1134); "Arch Dams: Measurements and Studies of Behavior of Kamishiiba Dam," by H. Kimishima and C. C. Bonin (Proc. Paper 1182); "Arch Dams: Construction of the Kamishiiba Arch Dam," by K. M. Mathisen and C. C. Bonin (Proc. Paper 1183); "Arch Dams: Review of Experience," by Robert E. Glover (Proc. Paper 1217); "Arch Dams: Stress

Note: Discussion open until June 1, 1957. Paper 1286 is part of the copyrighted Journal of the Power Division of the American Society of Civil Engineers, Vol. 83, No. PO 3, June, 1957.

1. Asst. Chf. Designing Engr., Civ. and Structural, Bureau of Reclamation, U. S. Dept. of the Interior, Denver, Colo.

Studies for Ross and Diablo Dams," by Joe T. Richardson and Owen J. Olsen (Proc. Paper 1267); and "Arch Dams: Economy of Concrete Dams," by Louis G. Puls (Proc. Paper 1286).

SYNOPSIS

Safety and economy are considered in the studies associated in the final selection of type for large concrete dams. Selection of type is further influenced by the arrangement of dam and appurtenant works. The paper concludes with pertinent unit cost data obtained from sources in various countries.

INTRODUCTION

Safety and economy are often considered together in the studies of Cost and Safety Criteria, but generally the two considerations are not inseperably associated in the final selection of type for large concrete dams. The safety of a large dam is the predominant consideration and is satisfied by criteria, which cannot be compromised by economic considerations. Important safety criteria are:

a) Suitable foundation conformation and strength to carry the imposed loads.

b) Stresses within allowable limits due to maximum design loads.

c) Satisfactory stability when subjected to maximum design loads.

d) Control to limit maximum design loads.

These are unalterable criteria, which relate wholly to safety, and there must be funds and benefits available to at least justify the construction of a dam satisfying these criteria or the project should be declared infeasible. The golden rule of dam design and construction is not to sacrifice safety to accommodate a budget.

After satisfying the safety criteria, plenty of latitude remains for the study of economic factors. The important economic considerations relate to structure planning, topography, hydrology, geology, type of dam, materials, accessibility, utility relocations, arrangement of appurtenant works, such as powerplant, spillway and outlets, construction methods, and many other considerations.

It has been stated in technical journals and papers, that American engineers prefer gravity dams and that our friends across the seas are further advanced in the knowledge of building arch dams. It is true that many straight or curved gravity dams have been built by the Corps of Engineers, the Tennessee Valley Authority, the Bureau of Reclamation, and others. These selections cannot be considered as a pronounced aversion to, or an indictment against, arch dams.

It is our view that there is no desire to fan the breeze of competition or ambition in dam building in either continent, and there exists solely cordial

relations with the mutual motive to interchange technical knowledge, to consult, and to assist one another to solve local problems and promote good will for the benefit of the profession and welfare of mankind in the development of natural resources.

Contrary to any imaginary belief, that the Americans prefer gravity dams, we subscribe to the selection of an arch dam at sites where this type can be used to advantage. In fact, the arch dam is preferred to the gravity dam because it not only requires less material and is therefore cheaper, but makes a much more effective use of material to provide much greater strength. A gravity dam may be called a monster structure with limited stability and surplus unutilized strength, whereas the arch dam may be considered an efficient embodiment of materials, strength, and beauty.

It was thought that a graphic picture might assist us in this discussion of economics of various sizes and types of concrete dams. The depiction attempt would necessarily be restricted to the important elements and could not be inclusive of all the properties and considerations, so caution must be exercised in the use of the information. The desired graph was obtained by correlating the three properties—canyon shape factor, height of dam, and volume—as shown in Figure 1. The selected canyon profile is a trapezoid for simplification defined by the dimensions, height of dam and developed length of crest, and the average slopes of the abutments. The formula for volume of dam has an approximate analytical basis through the process of integration, but is here considered hypothetical as a convenient means of creating a graph on which to plot data of dams, built or under study. The canyon shape formula resulted as one factor in the volume integration. The lower zone of the graph (small canyon factor) will relate to narrow canyons which would accommodate arch dams. The upper zone of the graph (large canyon factors) will relate to straight gravity dams. The middle zone would be a transition area and would relate to dams of varying curvature. The graph, based upon the full range of canyon shapes, is an attempt to depict by a family of curves, though approximate, the general American practice of concrete dam design for several past decades.

The number of dams over 100 feet in height, constructed or proposed in the world, of the several types within the last century, are approximately as follows:

Gravity	754
Arch	192
Multiple Arch	38
Buttress	39
Rockfill	77
Earthfill	358
Total	1458

Of this total, about 40 per cent have been constructed or proposed in the United States. Of all Nations, which have more than 10 dams, the United States has about 53 per cent of the total number; also, of all dams over 400 feet high in the world, constructed or proposed, about 40 per cent of them are located in the United States. These data have been taken from Mr. R. A. Sutherland's "Statistical Review," A.S.C.E. Proceedings, Separate 355, November 1953.

Comments may arise reflecting in some degree that the American practice is conservative. These comments are probably the result of hasty

observations. It is difficult to resolve to a comparable basis the several Countries' practices, because conditions vary—topography, geology, character, quality of materials, and consideration of the damage which might be done downstream of a site if the dam should fail. Cost of materials and labor and efficiency of doing work vary to the extent that hasty comparisons may lead to erroneous conclusions.

It is probably a fairly sound precept to use for guidance, that the evolution of dam design should be a slow process. This must necessarily be so, because a dam is a highly important structure regardless of size.

If the behavior, functions, and operation of a constructed dam conform to design criteria, assumptions, intentions, and expectations, all is well and a great source of satisfaction. A malfunction can cause serious concern and involve large remedial expenditures. A failure is a major disaster resulting probably in inestimable loss of life, property, and benefits many times greater in value than the original investment. To avoid the risk of failure, where consequences may be significant, is conducive to a long term evolution in dam design. The performance record of dams appears good, but a word of caution against overconfidence is urged in attempts to accelerate progress under the guise of mythical economy.

We believe our processes of trial load analysis or model testing for studies of stresses and stability are fully adequate, but we must not abandon continued search and study of theory and prototype behavior. We believe our knowledge of foundations is fully adequate but there may be additional important properties and characteristics of rocks yet to be revealed. We believe the materials are suitable, but frequently mystifying weaknesses do appear, urging further research. We believe our methods of construction are fully adequate but offer a fertile field for improvement to obtain better workmanship at lower costs.

A word of caution is urged in attempts to accelerate progress, in attempts to reduce thickness with increased heights of dam too far ahead of consolidation of gains in theory and firming up of new knowledge. We believe we are cognizant of the essentials of behavior of arch dams, but we cannot be too confident. There may exist, after a few years of duty, the gradual growth of a condition introducing serious deviation of anticipated behavior, which defies investigation either by trial load analysis or by model. Although much has been accomplished to improve the quality of concrete, deficiencies in durability have occurred for reasons unknown. Progress, yes, but cautious advances are advisable.

Structure planning is one of the most fascinating games of economics in dam design. Herein is exercised skillful selection of arrangement of the several structures and facilities to obtain the best operating conditions at the least cost. This process of choosing the most suitable position of the appurtenant works sustains inviolate the basic safety criteria of the principal structure, but does tolerate the risk of some damage under extreme operating conditions to minor elements depending upon the ease of repair, importance of outages, and calculation of long range economy.

Let us select, for discussion, two plans, with their variations, incorporating the gravity and the arch dam. Assuming the rock foundation is competent to support either dam and that the type of dam has been selected ideally based upon the canyon factor, that is, a wide river channel, indicates the selection of the gravity dam and a narrow canyon—the arch dam.

Now, the economic game starts. Our principal consideration is to choose

the best arrangement for the spillway, outlet works, and the powerplant. For the gravity dam we find the ideal arrangement is to spill over the dam, directing the flow through a dissipating works located in the river channel at the toe of the dam. The powerplant is located adjacent to the stilling works close to the dam. The river outlets are embedded in the dam and conveniently discharge on the spillway face into the stilling basin. The diversion of the river is accomplished by two stages; first, over blocks in the powerhouse area and, finally, over blocks within the spillway.

For the arch dam plan, the powerhouse is located in the channel as close to the toe of the dam as possible and the spillway is an inclined tunnel joined to a horizontal tunnel and discharges at a slight angle into the river channel at a suitable distance downstream from the powerplant. Diversion of the river is accomplished through a tunnel connecting to the horizontal reach of the spillway tunnel. The river outlets can be embedded in the dam and extended downstream past the powerplant discharging into the channel with maximum amount of dissipation obtainable.

The arrangements of dam and appurtenant works described are generally found to be most economical and provide suitable operating characteristics for the combinations, assuming the dam types have been selected according to their canyon factors. It is a rare occasion when it is possible to base economics upon types of dam only. The combination of all structures, dam, spillway, and powerplant must be considered together and the overall cost determined.

Suppose we are considering whether we should select the gravity or arch type at a specific site. This assumes that the canyon shape is suitable for the arch dam and, if so, it follows that it would probably also accommodate the gravity type. Now we are confronted with the difficulty of finding space to locate the spillway and powerplant for the gravity dam because the canyon is narrow. If the required spillway capacity is large, it will be desirable to discharge over the dam and place the powerplant at some distance downstream from the dam using the space in a natural widening of the channel or, if necessary, the provision for considerable excavation with extended penstock. If the spillway capacity is small, then there is economy in placing the powerplant in the channel at the toe of the dam and locating the spillway in a tunnel, which corresponds to the arrangement generally selected for the arch dam.

Diversion of the river during construction may be important in the selection of type of dam depending upon the magnitude of flow both in peak stage and duration. Shifting the river from one portion of the dam to another involves delays. These costs must be compared to tunnel diversion which permits continuous operations without river handling interruptions.

Comments may be of interest regarding spillway design. A gated or ungated spillway can be set in the crest of a gravity dam without difficulty but the thinness of an arch dam often introduces hydraulic and structural difficulties for an overflow crest arrangement. This difficulty can be overcome by grouting the contraction joints only to the elevation of the spillway crest and cantilevering the blocks above this elevation. For arch dams we prefer to construct the spillway independently of the arch dam structure.

Dissipation works are required only where there is need to avoid damage to the foundation of the important structure or impariment of draft tube operations. Discharge from tunnel spillways will generally cause erosion and create barriers in the channel resulting in reduction of power head. Channel maintenance costs from spillway action are part of the economic calculations.

Whether the spillway is conducted over a dam, through a tunnel, or released to fall freely gives great concern, especially for high dams, regarding serious detrimental action at the lower terminus of the flow. Misalignment of boundaries and flow lines may cause serious damage by cavitation. Joints or cracks in either concrete or rock are conducive to deflection of high velocity flow with creation of disruptive pressures or cavitation. The risk of subjecting foundation rock or protective slabs of concrete adjacent to the toe of the dam to disruptive force is an important consideration. Calculated risks should apply to secondary and minor structures but never to the integrity of the dam or its foundation in any attempt to obtain economy.

Due to progress brought about by experience, research, including studies of prototype behavior and thorough analytical processes, including photoelastic studies, our confidences have been supported to the extent that we have been agreeable to increasing permissible stresses in dams from 500 to 1,000 pounds per square inch in favorable instances within the last 25 years. The 1,000 p.s.i. permissible stress providing a safety factor of 4 to the concrete strength may be the limit we will hold to until our gains in knowledge of materials and behavior have been consolidated to give additional support to our confidences.

The American engineer and contractor constantly are on the alert endeavoring to find ways and means for reducing construction costs. Stimulated by the spiraling cost of living, demands for wage increases are ever before us. Because of increased efficiencies in plant construction and operation, the cost of materials and equipment has not advanced in the same degree as labor. With this changing ratio of labor over materials, labor saving methods in construction are given greater importance in this endeavor to reduce overall costs. One of the important elements of cost in a concrete dam is the cost of forms. Inasmuch as form construction consists mainly of hand labor, multiple use of forms is of prime importance in reducing cost of mass concrete in dams.

Unlike conditions that exist in many foreign countries, our labor costs are comparatively high and materials supply generally adequate. In many foreign countries the reverse is true; materials are scarce while labor is cheap and abundant.

For illustration let us consider a gravity dam, an arch dam, and a dome arch type of dam for the Flaming Gorge site. Comparative estimates have already been made for a gravity and arch dam at this site. The gravity dam will require about 1,900,000 cu. yds. of concrete and the arch dam will require about 940,000 cu. yds. of concrete. Even though the cost per cu. yd. for the larger volume will be less than for the smaller volume dam, the total cost for the gravity dam will be greater than for the arch dam. Where the dam abutments are competent and the crest length suitable, the arch type of dam would be adopted because of the substantial saving in overall cost. Suppose we now consider what effect a European dome-arch type of dam would have upon the cost at this site. It may be found that the dome type of dam will reduce the volume about 15 percent from the arch type. With this reduction the dome-arch dam will require about 800,000 cu. yds. of concrete as compared to 940,000 cu. yds. for the current American arch dam. On a volume basis, disregarding for a moment the more complicated form work required for the dome type dam and the limited reuse of forms, the cost per cu. yd. for the smaller volume will be greater than for the larger volume. When consideration is given to the additional cost for the more complicated form construction

and limited reuse thereof the costs in America of the dome-arch construction will be equal to or be greater than the arch type of construction. Essentially the difference of the two practices may be 15 percent in volume of dams, but when converted to respective costs the margin may be small, so that economic considerations may dictate different choice within the several countries.

Mr. Robert E. Glover has addressed inquiries to our friends in Italy, Portugal, and France on the subject of Costs of Concrete Dams. It was thought the interchange of specific information would assist in explaining the difference of practices. Gracious responses were received and furnish interesting data for our study and analysis. These replies are reproduced here in their original form.

a) Societa Adriatica Di Electricita, Italy, N. A. Biadene

"Further to our letter of August 18 and with reference to your letter of August 8 concerning the cost of concrete employed in European arch-dams.

"We were instructed by Mr. Semenzo, who is now taking a journey in the Far East, to supply you with all the information available on this subject—which unfortunately is rather limited.

"With regard to European costs in general, we regret we are not in a position to give you any details—with the exception of Switzerland where the price we believe is about 70 Swiss Francs, cement included.

"As for Italian prices, the cost of the concrete used in medium size arch-dams (about 75 000 cu.m. volume) built by our Company, is approximately 6 000 to 7 000 Lire per cu. m. cement excluded. The cost is, of course, higher for dams with smaller volume, and lower for those with greater volume. The incidences of labor in the making of concrete is about half the said figure. The incidence of materials amounts—approximately and with percentages varying according to whether it is river material or crushed rocks, distance from quarry, available transportation, etc.—to 10% - 15%. The remainder is constituted by working expenses and plant amortization.

"We cannot give you exact data concerning the other Italian dams not built by our Company, but assume they do not differ much from the above mentioned figures.

"We have good reasons to suppose that the cost of our concrete—in spite of pouring difficulties resulting from the particular shape (thinness, pronounced curvature, etc.) and the location of our constructions (often situated at great altitudes a.s.l., in cold areas, etc.)—may be regarded as rather low. This is due both to the relatively low cost of labor in Italy and to the keen competition among Italian builders that we consider very efficient and well equipped.

"We are sorry we cannot supply you with further details in this connection. Additional information on a collateral subject may be obtained from the Paper presented by Mr. N. A. Biadene to the Int. Comm. of Large Dams—5th Congress—'Considerations sur la comparaison entre les elements du prix de revient de quelques barrages italiens en beton,' that we send you under separate cover."

b) Laboratories Nacional De Engenharia, Civil, Portugal Manuel Coelho Mendes da Rocha.

"In replying to your letter of the 8th August 1956 I am sending you

herewith the requested information about the cost of the concrete in some of the Portuguese dams.

"The prices are given in escudos. To convert them into dollars the equivalence is 29 escudos per dollar:

	Year of the beginning of the works	Amount of cement (kg/m3)	Cost of concrete per m3 (escudos)*	Cost of cement per kg (escudos)	Cost of forms per m2 (escudos)
Castelo do Bode	1948	250	313	0,56	87
Venda Nova	1949	250	346	0,60	---
Cabril	1952	250	351	0,56	85
Salamonde	1952	250	337	0,60	---
Canicada	1953	250	345	0,60	---
Bouca	1954	250	372	0,69	50

* Cost of forms not included.

c) Andre Coyne & Jean Bellier, Paris, France
Symposium on Arch Dams—translation:

"We are answering your letter of 8 August 1956 in M. Martin's absence.

"The question of the data required on European concrete costs is a somewhat difficult one for the price per cubic meter depends on the costs of the auxiliary work such as excavation and formwork Contractors do not always distribute their remuneration in the same way under their various items. Their overheads and amortization of plant are not easily broken down with accuracy. In France, moreover, the question is complicated by the instability of the Franc.

"Here, however, are some prices for concrete in place in Arch Dams (including cement).

"(1) The cost given in F/m3 (Francs per cubic meter) is that at the date given in the relevant column.

"(2) The figure in brackets is the unit price.

"We also refer you to Report by Mr. Xerxes on Question 17 of the 5th Congress on Large Dams where you will find Portuguese prices:

Name of Dam	Volume (cubic meters)	Date (1)	Cost F/m3(2)	Remarks
La Chaudanne	27,000	1950	13,600	Comprising: formwork, plant, overheads Excluding: excavations appurtenant works
Bort	705,000	1950	12,800 (4,750)	Comprising: Formwork, plant, overheads Excluding: excavations appurtenant works.

Name of Dam	Volume (cubic meters)	Date (1)	Cost F/m3(2)	Remarks
Enchanet	66,000	1952	6,900 (4 300)	Comprising: Formwork, excavations Excluding: appurtenant works, plant, overheads
Chastang	250,000	1948	8,000	Comprising: formwork, plant, overheads, excavations Excluding: appurtenant works
Tignes	630,000	1952	9,500	Comprising: Formwork, plant, overheads, excavations Excluding: excavations appurtenant works
Castillon	125,000	1950	15,000	Comprising: Formwork, plant, overheads, excavations Excluding: appurtenant works
Rassisse	12,500	1954	14,000 (7,300)	Comprising: Formwork, plant, overheads, excavations Excluding: appurtenant works.

Contract bid prices indexed to January 1952 for mass concrete of several dams in the United States are shown in Table I.

The unit costs of mass concrete in several countries of volumes less than 1,000,000 C.Y. based upon information described above are as follows:

Country		Unit Cost including cement except as noted $/C.Y.	Exchange rate per dollar used
Switzerland		12.50	4.28 Swiss Francs
Italy	(1)	8.00	620 Lire
Portugal	(2)	9.00	29 escudos
France	(3)	10.40	350 Francs
Australia	(4)	17.81	0.447 pounds
United States		15.60	

(1) Cost of cement not included

(2) Cost of forms not included

(3) May be a misunderstanding of information due to translation. Assumed that cost of Bort Dam concrete of 4,750 Francs per cubic meter was correct.

(4) Cost of cement not included (Tumut Pond Dam, 170,000 C.Y.)

CONCLUSIONS

The above limited data indicate tentative conclusions as follows:

a) Unit cost of mass concrete in the U. S. is higher, including both labor and materials, than in Europe.

b) Amount of cement per unit volume used in European dams is greater than in the U. S.

c) Possible efforts to reduce unit costs in the U. S. offer slight hope of improvement:

1. The present use of about 280 pounds or less of cementing materials per c.y. is minimum to obtain satisfactory quality.

2. In view of present policy to design for maximum stress of 1000/lbs.-sq.in., where applicable, any further reductions in volume would induce higher unit prices and tend to compromise factors of safety.

3. The dome type of dam, generally used in Europe, would probably demand higher unit prices than the vertical arch type used in the U. S. where labor prices are high and are not receptive to additional complicated formwork.

TABLE I COST FOR MASS CONCRETE IN DAMS (U. S. BUREAU OF RECLAMATION)

Year	Dam	Volume C. Y.	Unit Price $/C.Y. Low bids			Unit Price $/C.Y. Indexed to Jan. 1952		
			Excluding Cement	Cement	Total	Excluding Cement	Cement	Total
1931	Hoover	3,400,000	$2.70	$1.60	$4.30	$8.20	$3.95	$12.15
1934	Grand Coulee	3,100,000	3.00	1.90	4.90	8.35	4.25	12.60
1937	Grand Coulee	5,500,000	3.53	2.07	5.60	8.26	4.24	12.50
1938	Shasta	5,400,000	4.00	1.95	5.95	9.36	3.99	13.35
1939	Friant	1,850,000	2.48	1.97	4.45	5.80	4.00	9.80
1946	Kortes	130,000	10.00	3.75	13.75	15.90	4.50	20.40
1948	Hungry Horse	2,900,000	7.00	3.50	10.50	8.40	3.85	12.25
1949	Canyon Ferry	390,000	11.00	3.00	14.00	12.65	3.25	15.90
1953	Monticello	260,000	12.90	3.20	16.10	12.25	3.15	15.40

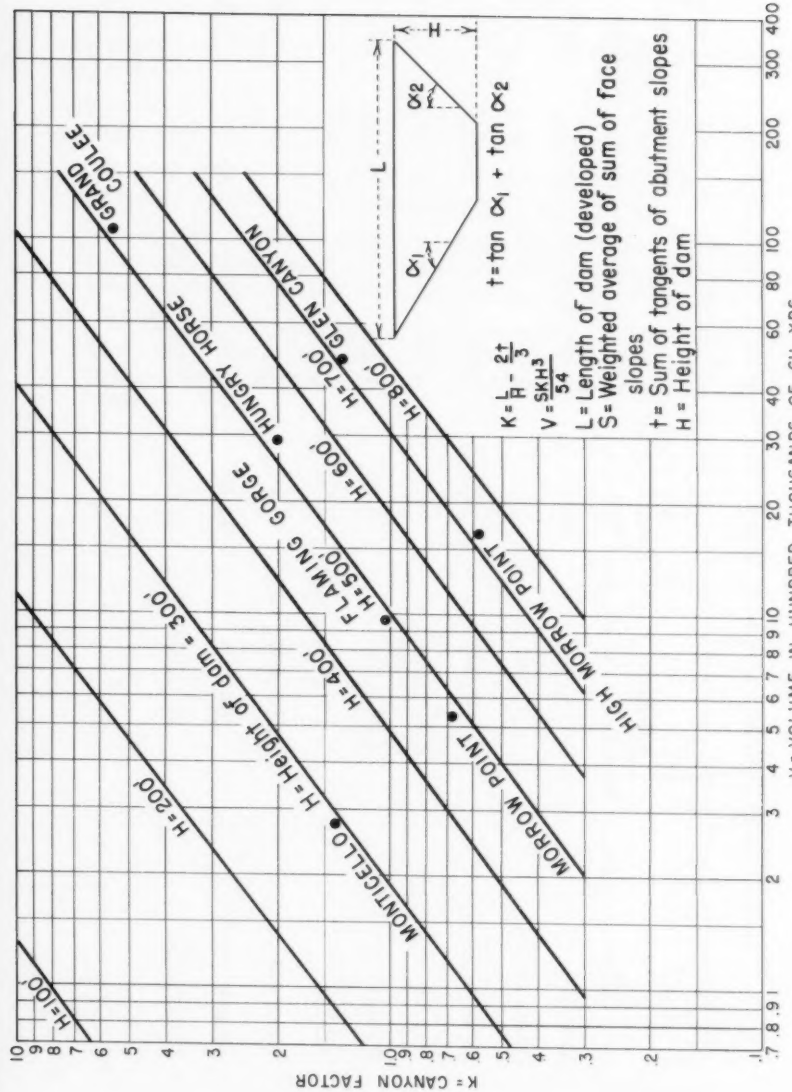

ESTIMATED VOLUME OF CONCRETE DAMS
V = VOLUME IN HUNDRED THOUSANDS OF CU. YDS.
K = CANYON FACTOR
K = L/R - 2t/3
V = SKH³/54
L = Length of dam (developed)
S = Weighted average of sum of face slopes
t = Sum of tangents of abutment slopes
H = Height of dam
t = tan α1 + tan α2
H = Height of dam
GRAND COULEE
MONTICELLO
FLAMING GORGE
HUNGRY HORSE
GLEN CANYON
HIGH MORROW POINT
MORROW POINT
H=100'
H=200'
H=300'
H=400'
H=500'
H=600'
H=700'
H=800'

Figure 1.

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Note: Paper 1287 is part of the copyrighted Journal of the Power Division of the American Society of Civil Engineers, Vol. 83, PO 3, June, 1957.

Discussion of
"ARCH DAMS: THEORY, METHOD AND DETAILS OF JOINT GROUTING"

by A. Warren Simonds
(Proc. Paper 991)

A. WARREN SIMONDS,[1] M. ASCE.—The discussion by Mr. Glover contains some interesting observations relating to the thermal conditions of arch dams in regard to thickness of the section. While most arch dams constructed prior to 1930 were allowed to cool naturally, it was always difficult to find a time when the optimum temperature prevailed throughout the structure. The minimum temperature of the mass concrete in a dam occurs in the late winter or early spring season. In the upper part of the structure, where the section is thin, the concrete will reach its minimum temperature while the concrete is still cooling in the thicker part at the lower elevations. It was therefore necessary to select a time for grouting the joints when a more or less "average" temperature prevailed. By installing an embedded system of coils throughout the concrete and cooling the concrete artificially, not only is the setting heat of the concrete removed at an early date without regard to the seasonal variation in temperature, but also better control of the temperature distribution is obtained.

Mr. Elston mentions the possibility of creating an "arched" monolith by grouting the contraction joints. A concrete arch dam should be monolithic for the most desirable distribution of stress. The effect of the grout pressure in the contraction joints may be considered as "prestressing" because the distribution of stress is changed by the grout pressure in the joints. This phenomenon has been indicated by observations on strain meters embedded in the mass concrete and also by precise survey measurements which have shown that the position of structure has been moved as a result of the grout pressure.

Grout pressure in the contraction joints, if properly balanced will place the blocks in the arch ring in compression. The only tension that will develop, as a result of grout pressure, will be in the cantilever elements. When the load comes on the structure, the deflection and the stress distribution will depend on the geometry of the structure, that is the thickness of the arch, radius of curvature, central angle, and also to possible changes in temperature. In a properly designed arch dam, the stresses should be compressive, but if the topography of the site and the configuration of the dam are such that some tension should be inevitable, the tension should be held to a minimum. In most arch dams a downstream deflection will cause a compressive stress to predominate at the abutments. This may be maximum at the downstream face and diminish toward the upstream face or even becoming tensile at the upstream face. For long slender arches, downstream deflection may occur at the crown and an upstream counter flexure at the quarter points nearest the abutments. When this occurs, the stress distribution at the ends of the arch is the reverse from that previously described.*

1. Engr., Bureau of Reclamation, U. S. Dept. of the Interior, Denver, Colo.

* Report on Arch Dam Investigation, Engineering Foundation, Vol. I, 1928. Model Tests, Analytical Computation and Observation of an Arch Dam ASCE Separate 696.

The general practice of the Bureau of Reclamation is to use keys in the joints of all solid concrete dams, of both arch and gravity types, regardless of whether the joints are to be grouted or not. In the case of straight gravity dams, where each block or monolith is statically stable and will carry its load independently of the other blocks of the dam, keys are not essential if the joints are not to be grouted.

In regard to the partial filling of the reservoir to balance the resultant of grout pressures in joints, such a condition involves some hazard if the stream is subject to flash floods, in which case, the reservoir water load would be carried by an ungrouted arch. It is not necessary to raise the reservoir water surface to exactly the same level at the same time fluid grout is being injected in the joints. The elevation of the reservoir water surface should be determined within limits which will prevent excessive tensile cantilever stresses from developing in the blocks of the dam. Upstream deflections of small magnitude do not appear to be harmful if they are not accompanied by bending stresses which are likely to cause tensile cracking at the downstream face, or splitting the grout film in the previously grouted lower lifts of the joints.

For ideal conditions, grouting lifts of lesser height than 50 feet, as suggested by Mr. Elston, are desirable from a stress standpoint. From a practical standpoint, the higher grouting lifts which can be used satisfactorily will reduce the total number of lifts in the structure to be grouted and will, therefore, make the grouting operation less costly. Grouting lifts have been used having heights ranging from 25 feet in a few cases to 125 feet; the height of 50 feet has been selected as being practical and economical for most arch dams.

Discussion of
"THE EFFECT OF PORE PRESSURES ON STRESSES IN GRAVITY DAMS"

by O. C. Zienkiewicz
(Proc. Paper 1042)

O. C. ZIENKIEWICZ.[1]—The author would like to express his thanks to Mr. Park for his contribution and the re-statement of his equations 13 & 14 in a new form based on less restrictive assumptions. The uncertain value of the Poissons ratio coupled with only approximately elastic behaviour of concrete led the author to make the assumption that the Poissons ratio could be neglected. It is useful however to have the full solution and to note that the basic conclusions derived in the paper are materially unaltered even if a value of the Poissons ratio as high as 0.15 is assumed.

CORRECTIONS.—On page 4, in Eq. 2a, the first term $\frac{1}{r}\frac{\partial \delta r}{\partial \theta}$ should be changed to $\frac{1}{r}\frac{\partial \delta \theta}{\partial \theta}$.

On page 4, in the line under Eq. 2a, the word "plain" should be changed to "plane."

On page 12, in the second equation of Eq. 15 the last term n $\cos\alpha \sin\beta$ should be changed to n $\cos\beta \sin\alpha$.

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PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Board of Direction are identified by the symbols (BD). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper numbers are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 1113 is identified as 1113 (HY6) which indicates that the paper is contained in the sixth issue of the Journal of the Hydraulics Division during 1956.

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c. Discussion of several papers, grouped by Divisions.

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